PSDC - TR - 18

A STUDY FOR THE DEVELOPMENT OF A STATIC LOAD TEST FACILITY TO BE ESTABLISHED AT THE PROTECTIVE STRUCTURES DEVELOPMENT CENTER FORT BELVOIR, VIRGINIA

FINAL REPORT

APRIL 1966

SPONSORED BY

OFFICE OF CIVIL DEFENSE

PREPARED BY

WISS, JANNEY, ELSTNER AND ASSOCIATES
DES PLAINES ILLINOIS

CONTRACT NO. DA-18-020-ENG-3566
OFFICE OF CIVIL DEFENSE WORK ORDER OS-63-148
SUBTASK 1127 C

FOR THE

PROTECTIVE STRUCTURES DEVELOPMENT CENTER

FORT BELVOIR, VIRGINIA

A BRANCH OF THE JOINT CIVIL DEFENSE SUPPORT GROUP

OFFICE OF THE CHIEF OF ENGINEERS/NAVAL FACILITIES ENGINEERING COMMAND

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SUMMARY

A STUDY FOR THE DEVELOPMENT OF A STATIC LOAD TEST FACILITY TO BE ESTABLISHED AT THE PROTECTIVE STRUCTURES DEVELOPMENT CENTER, FORT BELVOIR, VIRGINIA

This report contains a review of the state of knowledge concerning the inelastic behavior characteristics and ultimate load capacity of reinforced concrete floor and roof systems. The yield line theory normally predicts an ultimate load that is less than tests show for systems that are restrained at their perimeter.

There have been very few tests of full-scale buildings to the collapse stage. Most of what we know today about behavior of two-way floor systems has been based on structural systems tested at less than full scale. This report concludes that less than full-scale testing is entirely practical and desirable. Models scaled down to 1/28 are feasible, but the best scale seems to be between 1/8 and 1/15. The report discusses considerations that must be given to materials, fabrication and loading techniques in setting up a model test program.

The report outlines a program in detail to test three-dimensional frameworks of conventional reinforced concrete floor and roof systems. The first step is to establish proper correlation between mortar models and full-scale prototypes on their principle behavior considerations; that is, shear, flexural strength, bond, and column action. A series of tests on simple components is outlined to study these considerations. After the correlation problems have been resolved with mortar models of single elements, the program of testing conventional three-dimensional frameworks can be carried out. Typical building frames are detailed for such typical floor systems as flat slab, flat plate, two-way slabs, pan joists systems, waffle slabs and precast prestressed concrete.

The report also contains a bibliography of published articles dealing with ultimate strength of two-way and three-way systems and the various theories that have been proposed. Also included is a review of papers dealing with structural model analysis.

FOREWORD

This report has been prepared under Contract No. DA-18-020-ENG-3566 (24 January 1965) as part of a "Study for the Development of a Static Load Test Facility to be Established at the Protective Structures Development Center, Fort Belvoir, Virginia." The material covered herein is intended to accomplish the tasks outlined by the PSDC Schedule for Work Unit OCD-PS-65-17, Subtask 1127A. The report has been prepared in two sections, Parts I and II, as follows:

Part I

Prepare a study for use by the Protective Structures Development Center in the development and establishment of a Static Load Test Facility. The facility shall be capable of supporting a long range program for evaluation of the behavior of conventional reinforced concrete structural systems through correlating the behavior of models and prototypes under static loads to collapse load. The study shall be the result of an investigation and evaluation which shall include, but not necessarily be limited to:

1A. A review of the state of knowledge of load-deflection characteristics of reinforced concrete floor and roof systems in the inelastic range to collapse load with emphasis on studies verified by experimental evidence.

2A. An investigation of the suitability of models, including special requirements of model materials, scales and instrumentation for evaluation of the structural behavior of full-scale reinforced concrete systems in the inelastic range under statically applied loads.

3A. An investigation of loading methods compatible with the requirement for evaluation of model and full-scale system behavior in the inelastic range. The investigation shall include an evaluation of each method based on cost of initial installation, subsequent operating and maintenance costs and personnel requirements for operation. Loading methods to be evaluated shall include, but not be limited to: air evacuation (vacuum), air overpressures, fluid loading, mechanical jacking systems and combinations thereof.

4A. An investigation of the characteristics of reinforced concrete systems pertinent to defining failure on the basis of ultimate load carrying capacity.

Paragraphs 1A and 1D have been grouped together and are presented under the heading, "Load Carrying Capacity of Reinforced Concrete Floor and Roof Systems." Paragraphs 1B and 1C have been grouped together and are presented under the heading, "Models".

FOREWORD

Part II

1B. Recommended model scales, materials and instrumentation to accomplish the evaluation of conventional reinforced concrete systems through correlating the behavior of models and prototypes under static load to collapse loads. The recommended scales, materials and instrumentation shall permit reliable determination of the behavior of the structural system through the entire range of loading to ultimate structural capacity.

2B. A description of the recommended methods of loading the models and prototypes, accompanied by conceptual drawings of the method or methods, in sufficient detail to permit determination of its suitability by the Contracting Officer.

3B. Types of systems and structures to be evaluated.

4B. Conceptual views or line drawings of the test facility indicating location of model and prototype test stands, location of control array for monitoring and operating the instruments and equipment, and fabrication and material storage areas.

5B. A plan for operating the facility including personnel requirements and organization.

6B. Itemization of instruments, equipment, materials and supplies required to operate the facility.

7B. A recommended long-range experimental program to be undertaken at the facility for evaluation of the inelastic behavior and ultimate structural load-carrying capacity of structural systems. The outline should be organized in sequential phases so as to maintain a continuing program of testing and evaluation. An estimated time for completion of each phase shall be provided.

The information on which this report is based was gathered from the following sources:

A library search and review of significant literature pertaining to the subjects of mortar model studies and failure criteria for reinforced concrete floor and roof systems.

Personal contact by telephone and letter with four of the leading model analysts who are in charge of large model testing facilities in Europe.

Personal contact with persons in the United States who we believe are most knowledgeable in the area of reinforced concrete research

and mortar model testing.

Our own background as a result of ten years experience in the field of small scale structural model investigations.

A selected, annotated bibliography containing a literature review of significant published and unpublished papers and books which have come to our attention during this study is provided. In addition, the replies received from the European authorities have been reproduced verbatim and are included in an appendix.

CONTENTS

PART I

CHAPTER 1

LOAD CARRYING CAPACITY OF REINFORCED CONCRETE FLOOR AND ROOF SYSTEMS

1.1 SUMMARY OF PRESENT STATE OF KNOWLEDGE

Before outlining an experimental program for studying the total strength of reinforced concrete buildings, we will summarize and evaluate the present state of knowledge on the subject. Without this review, a program which will yield new information of practical value cannot be formulated effectively.

In general, a great deal is known about the ultimate strength and deformation characteristics of individual reinforced concrete units, which we will term one dimensional elements. For example, theory on the ultimate capacity of axially-loaded reinforced columns and columns under combined axial load and bending is well established. For short columns there is little discrepancy between theory and actual test results. There is probably still a need for further experiment on long columns but it is doubtful that future tests will disclose important differences from the more sophisticated analytical methods that are presently available. The ultimate flexural strength of beams is very well known for both reinforced concrete and prestressed concrete. Knowledge of the shear strength of one-dimensional members is also in an advanced stage. Modern theories on shear strength consider the effects of flexural tension and consequently apply well for both positive and negative bending. Perhaps less well understood in the study of shear is the strength reducing effects of concrete shrinkage when the concrete is internally or externally restrained.

When concrete building units are arranged or cast together into frameworks lying in a single vertical plane (two-dimensional systems), we enter into an area where a little less is known because considerably less research and testing has been accomplished. Still, by applying the principles of limit design in which the redistribution of moments are recognized, and by making use of our knowledge of the behavior and strength of individual units, we can predict collapse loads quite accurately. If a three-span bent complete with colums were loaded to failure, the results would probably not be surprising when compared with predictions based on existing theory. Even if the steel were varied within the members for several such bents, limit design theory would be close to the mark.

A simple span beam designed by elastic theory could be assumed to have a safety factor of about two if subjected to an isolated load test. But if we were to make the same assumption for a two-way slab designed in accordance with past and present code methods (elastic), we would probably be grossly in error.

1.1.1 Three Dimensional Frames. It is when concrete assemblies become three-dimensional with the loads applied principally to elements in horizontal planes and transferred through the horizontal plane system to other supporting members that accurate strength predictions become difficult. Tests on such units have resulted in considerably greater safety factors.

When horizontal frames are connected to vertical frames, a configuration results that has received little experimental study. Full-scale three-dimensional frameworks such as typical reinforced concrete building frames have rarely been loaded to failure. We know of only a few such cases. Professor A. J. Ockleston of the University of Witwatersrand conducted a number of destructive tests in 1955 on the Old Dental Hospital in Johannesburg, South Africa (16A, 17A, 18A, 19A)[1]. The results were rather startling. For the case of a two-way slab, the applied test load was 15-1/2 times the super-imposed load for which the floor was designed. The total load (843 lbs./sq. ft. including slab dead weight) was 8 times the total design load. Another floor system was tested to failure in 1962 on a full-scale mock-up for the Place Victoria in Montreal (20A). In this test, two different designs for shear were compared. The ACI design method for shear proved the strongest of the two, but the system still failed in shear at a load of 322 lbs. per sq. ft. Complete collapse did not result from this test. In neither the Canadian nor South African tests was the loading continued to complete collapse.

A three-quarter scale, nine-panel flat plate, was tested to failure at the Portland Cement Association Structural Laboratory in 1962. This structure failed by punching shear at one of the interior columns at a total load (DL + LL) of 369 lbs. per sq. ft. (21A).

A shell roof could be considered a three-dimensional framework and a few mortar shells have been built and tested abroad and in this country.

1.1.2 Loading in Plastic Range. Two-way reinforced concrete floor and roof systems have been given a great deal of study. In spite of the amount of work that has been done on slabs, their behavior is still not completely understood at advanced stages of loading. Timoshenko's (9B) basic work covers reinforced concrete plates and slabs in the elastic range. Stress and deformation predictions thus derived have proven quite accurate within the range of elastic response when the boundary conditions are properly established. W. Prager (10B) and others have studied plates in the plastic range, and have attempted to apply plastic theory to reinforced concrete. These efforts have been only partially successful. Plastic theory assumes that collapse will occur when the strains or stresses within a plate exceed the limits set forth in one of the various theories of failure. Among the theories used for determining end points

[1] Numbered reference in selected bibliography.

are the square yield criteria, the von Mises yield condition (oblique elipse), and the maximum shear stress or Tresca's criterion (oblique hexagon). Johansen (8B) points out:

"Any theory of failure must be based on a hypothesis of failure which in association with certain geometrical assumptions. leads to the determination of the state of stress. Even the simplest of all failure hypotheses, such as Guest's hypothesis or the law of friction for earth pressure, involves complicated differential equations. The law of superposition does not apply, and so these equations are not linear either. Consequently, only relatively few of the cases encountered have been soluble".

This, coupled with the fact that no theory of failure applies exactly for the semi-brittle, semi-plastic material of reinforced concrete, seems to explain why plastic theory has not been very useful for predicting the ultimate strength of concrete slabs. Johansen developed a simpler approach for ultimate design of slabs in 1931, which extended a theory originally introduced by A. Ingerslav (22A, 23A) in 1921. Johansen points out that in a plastic material, yielding can occur in two very different ways: (1) three-dimensionally or (2) along yield surfaces. He says further, "this could also be expressed by saying that in the first case, yielding spreads along a system of surfaces infinite in number and in the second case, only along some quite specific yield surfaces". Johansen's yield-line theory thus presumes yielding will take place along certain definite lines where all rotation is accumulated, and the portions between yield-lines remain rigid planes within which internal stresses can be ignored. It is generally recognized that to ignore stresses (strains) in the planes between yield-lines is a simplification which should over-estimate the strength of a slab. For this reason, plastic theory methods where differential equations for analyzing the forces on the infinite number of planes and setting these to some theory of failure endpoint are often called lower bound solutions. The yield-line theory methods are termed upper bound solutions. Simplified techniques have been developed for lower bound solutions such as those proposed by Hillerborg (6A, 7A). Experimental work on reinforced concrete and mortar slabs has failed to show that the yield-line theory is, in fact, an upper bound solution. In 1953 Hognestad (24A) said:

"It may, nevertheless, be generally stated that tests verify that the yield-line patterns are a reality, and the ultimate loads predicted by the yield-line theory are on the average 80 to 90 per cent of those observed in tests. Although the yield-line theory might be expected to give an upper limit for the ultimate load as full plasticity is assumed in the yield-lines, an additional strength is observed in tests. This is believed to be primarily due to strain hardening in the reinforcement and to the presence of membrane action at the relatively large deflections near failure". Hognestad has continued to follow this subject and in a recent interview, reiterated that he still believed the yield-line theory to be conservative.

In some cases, the yield-line theory has indicated failure loads less than half the actual ultimate load. Secondary effects, such as compressive dome action or tension membrane action, have greatly enhanced the collapse strength. It may therefore be assumed that as a practical matter, the yield-line theory is a lower bound solution if sufficient strength exists to permit loads to be transferred to supporting elements and downward without failure of any support.
Wood (7B), explains in concluding his book:

"It may now appear that some of the intricate points of theory were hardly worthwhile in view of the doubtful yield criteria, and of membrane action, and of the test results. A typical instance is the proof in Art 46b that the collapse load for a clamped square slab may be as low as 37.7M.

"But the details of plastic theory have been included deliberately, in order to prevent the idea gaining ground that simple plastic analysis per se is all that is required; and, above all, to ensure that a careful treatment will be made when more realistic, and less conservative yield criteria are discovered".

It is unwise to ignore the plastic theory approach in designing or analyzing reinforced concrete slab systems. Eventually, a general plastic theory will probably be developed which will apply well for reinforced concrete. At the present, however, the yield-line theory is the most convenient and accurate method for analyzing reinforced concrete slab behavior.

1.1.3 Dome and Membrane Action. Recently investigators have started to study quantitatively dome action and membrane action. Ockleston first looked into "arching" or dome action after he was puzzled by the fact that the yield-line theory predicted only 39% of the test load for the two-way slab in the Old Dental Hospital (19A). Later, in 1958, he formulated a method for analyzing what he calls arching but admitted that his approach would not predict the failure load. Wood (7B), devotes one chapter of his book to this phenomenon, calling both in-plane compression and tension membrane action. Wood points out that because of in-plane compression, the yield moments can be almost halved, which explains why yield-line theory may miss test results by a factor of two. Wood also points out that after crushing in dome action, the slab deflects further to form a tensile membrane. In December, 1964, a paper was published in Germany entitled "Dome Action in Continuous Reinforced Concrete Slabs" (43A). The paper reports tests made on 0.4 scale models conducted at Stuggart Technical University. It is obvious that in order for either type of in-plane stress condition (compressive or tensile) to develop, a tension or compression resistance must be available at the periphery. For dome action the German paper states that the amount of thrust that is to be resisted depends upon the load, stiffness of the tension ring, Poisson's ratio, modulus of elasticity, height of the resultant force, the slab thickness to span ratio, and the short-span to long-span ratio.

It follows from all of this that the strength enhancing effects of in-plane forces within the slab can only be counted upon in interior bays where the necessary restraint can be provided by the surrounding bays. Compression and tension ring action is also possible for exterior bays if the spandrel beams are sufficiently stiff. Wood again cautions "no allowance for membrane action should be made if the supporting beams are involved in the mechanism of collapse".

1.2 COLLAPSE LOADS OF REINFORCED CONCRETE BUILDINGS

Using the foregoing as background, how would we expect typical reinforced concrete buildings to fail under heavy over-pressure loading? What information are we missing? Are there any strength enhancing features in typical buildings which might tend to overcome the points of stress already noted in the laboratory.

We can perhaps make some speculative answers to these questions by considering the following general categories of reinforced concrete structural frames which have been built in the past fifty years:

1. Flat plate. No drop panels at columns and no supporting beams except perhaps at the perimeter of the building.

2. Flat slab. Same as flat plate except with drop panels at the columns.

3. Two-way slabs. Slabs which are supported on all edges by means or walls framing between columns.

4. Waffle slab. Ribbed slab's spanning in two directions.

5. One-way slab and beam. Flat slabs supported on two edges only either on beams, girders or walls.

6. One-way pan joist. Ribbed slabs spanning in one direction with or without nominally reinforced diaphragms.

7. Soffit block systems. Soffit blocks of concrete or tile which serve essentially as fillers between ribs.

8. Precast systems.

   a. Precast reinforced or prestressed members arranged as beam and post constructions.

   b. Composite and continuous precast construction.

9. Shells. Roof structures where strength is derived from the three dimensional geometry.

Let us take a look at the above categories in the order listed.

1.2.1 Flat Plates. Flat plates are most common method of constructing modern apartment buildings. The flat plate is favored because of low cost, simplicity, and speed of construction, and the smooth lower soffit provides for an inexpensive contact ceiling. Most apartment buildings built in Northern climates have curtain wall construction of brick, concrete, or steel panels. These curtain walls usually hang from the frame with insert devices which will not allow transfer of load from the floor system to the wall; hence, the walls probably provide little resistance to vertical loads. In situations where the frames are exposed on the surface with masonry filling in the rectangular column-spandrel beam grids, substantial strength is no doubt afforded by the walls for both vertical and lateral loads. Internally, the partition system which bears between floors will probably strengthen the building considerably. Some of the more recent types of partitions, however, are not very stiff and their contribution should not be over-estimated. Flat plates often have slender spandrel beams in modern architecture so the strength of the floor system in the exterior bays may not be greatly enhanced over yield-line predictions if auxiliary support does not exist. The absence of drop panels in flat plate construction makes this system most vulnerable to punching shear (21A). The connection between the slab, spandrel beam and column is another vulnerable spot as pointed out by the University of Illinois mortar model tests.

1.2.2 Flat Slabs. The flat slab floor system is not well suited for apartments because of the drop panels but has been used extensively in the past for industrial buildings, stores, warehouses and parking garages. It is still quite prevalent for the latter two building types. The presence of drop panels greatly reduces the probability of punching shear failure and provides an opportunity for flexural failure at loads equal to or above those predicted by yield-line theory. The slab to exterior column connection is still vulnerable and failure can be expected here in shear, spandrel beam torsion or column flexure before the slab itself collapses.

1.2.3 Two-Way Slabs. Two-way slab systems were widely used in the past but appear less frequently in present day designs. The two-way slab floor undoubtedly possesses great reserve strength as demonstrated by Ockleston and at Stuggart. The presence of the beams greatly increases the system's resistance to punching shear at the columns. The exterior beams would still be vulnerable to torsion failure as was noted in the deep spandrel beams at Illinois. Breaking off of the exterior columns at the base of the spandrel beams may also occur at very heavy loads.

1.2.4 Waffle Slabs. Waffle slabs can be expected to behave like flat plates, and the presence of the ribs will not likely affect the yield-line patterns greatl· The weaknesses previously noted for flat plates should apply here. Waffle slab construction may not employ floor to ceiling partitions because when partitions are required, it is customary

to use hung ceilings with waffle slabs. Partitions thus frame into the ceiling system and consequently, no strengthening effects can be expected.

1.2.5 One-Way Systems. One-way slab and beam systems are not employed too frequently in modern practice as the basic structural system. However, this scheme has been used in the past and many structures, especially smaller ones, are in existence which have been so constructed. Even through this slab is designed as a beam strip, we would expect considerable two-way action due to the action of non-structural supporting elements, such as partition walls and other boundary conditions.

One-way pan joist systems are designed to carry loads in one direction only. It is common practice to install diaphragms between the joist ribs and to provide heavy spandrels along the column lines and at the edges of the building parallel to the ribs. Because of the presence of the diaphragms and the parallel spandrel beams, these systems may be expected to exhibit a considerable amount of two-way action. We have observed this two-way action in model studies using plastic. We also noted two-way action in a mortar model of a one-way slab system which was tested at Princeton University. In this model test, yield-line patterns developed very similar to the patterns that would be expected for a typical two-way flat slab. Two-way action may not develop significantly in those systems where the spandrel beams are omitted and in cases where the diaphragms are not employed.

Soffit block systems will probably behave very similar to one-way pan joist systems. The presence of the concrete or tile fillers probably strengthens the system considerably in the elastic range but may not be of any great help for loads approaching collapse. In modern soffit block construction, it is usual to use contact ceilings and run the partitions from floor to ceiling. Consequently, the partitions may be of assistance in strengthening the floor system.

1.2.6 Precast Construction. Of all of the reinforced concrete systems considered, precast beam and post construction is probably the weakest. Precast structural elements in which composite construction is not employed are usually assembled as individual units with meager devices employed to tie the units together. These devices may be cast-in weld plates or shear keys in which grout or concrete is placed. These members will, therefore, fail under heavy loads as the load is applied to each element directly. The state of knowledge of the capacity of these individual precast units is well established with respect to flexure and shear. Collapse, however, will seldom occur with the failure of the member in either of these two modes. Most collapses will be the result of either bearing failure or excessive rotations at the support. The failure mechanisms of bearing and rotation are not well established, and little work has been done in this area. We have observed the results of loads producing deflections in excess of two or three feet in precast, prestressed elements without complete collapse. Collapse did occur, however, when the end rotations reached enough magnitude to translate masonry bearing walls or supporting

beams sufficiently to permit the members to drop at the end. It might be concluded then that precast construction of the beam and post type will act essentially in one direction with the effect of load transfer in the lateral direction being a minor factor.

The type of precast construction employing composite action is substantially stronger than simple precast framing. In this case, we are referring to construction where the precast elements are tied together with the use of cast-in-place concrete. In some cases, continuity is established through the cast-in-place sections. When precast concrete is adequately tied together with the use of cast-in-place concrete, the total structural system can be expected to act very similar to monolithic reinforced concrete construction (44A, 45A).

1.2.7 Shells. Shells are usually designed so that the principal load-carrying capability is through in-plane compressive or tensile stresses flowing through the shell to the supporting elements. If the supporting elements. If the supporting element has sufficient stability, most shells fail eventually in buckling. In many cases, however, shell structures are supported on elements which in themselves are not capable of sustaining excessive loads. These heavy loads may produce large rotations or deflections and may be substantially weaker than the shell itself. Much of the testing which has been conducted on mortar models has been simplified so that the supporting elements are considerably greater in stability and strength than those which would exist in the prototype. It appears that effective studies in this area using small-scale mortar models should take into consideration the supporting elements and be sufficiently three-dimensional in scope to produce behavior through supports which are representative of actual cases.

CHAPTER 2

DEFINING FAILURE FOR REINFORCED CONCRETE FLOOR AND ROOF SYSTEMS

2.1 GENERAL

It is obvious that the word failure means different things to different people when the performance of a structural system is evaluated. A cracked floor, ceiling or wall might be considered failure to the resident of an apartment. An architect might consider a floor system which has deflected a considerable amount to have failed. He could contend that due to the excessive deflection, the floor system was no longer performing the function for which it was intended. This definition of failure is quite often used.

2.1.1 Functional Limitations. The problem, of course, is defining the point at which the system no longer serves its intended function. An engineer performing research on structures or structural components might define failure as the point where the structure no longer can sustain an increasing load. Most researchers consider failure of the structural system in much the same way that we normally think of failure in testing a 6" x 12" concrete cylinder. In testing concrete cylinders failure is considered to have been reached at the maximum load indicated by a load follower on the machine. Although it is much more difficult to pinpoint failure in a structural system, the analogy to the concrete compression tester is consciously or unconsciously made. It is probably for this reason that almost all of the tests made in the laboratores and in the field that have been reported in the literature were stopped when it appeared the structure could no longer sustain an increasing amount of load.

2.1.2 Load Carrying Capacity. In most of the cases reported it is safe to conclude that the structural systems could have deformed a great deal more before total collapse. Although the maximum load may have been reached, the area under the load-deformation curve beyond the maximum is very important in considering the total energy absorbing capacity of the structural system.

A structural system that has completely collapsed to the ground would satisfy any definition of failure. A structural system that had deformed so much that it is no longer possible to occupy the contained space no matter how primitively would also satisfy almost any definition of failure. It is perhpas this latter definition that comes closest to the defining failure for purposes of blast protection.

2.1.3 Deflection. It is common to think of structural systems as being quite ductile and to visualize failure in terms of excessive flexural deflection. If failure is to occur by this mode, then we could consider

a system as failed when the deflecting system almost touches the underlying level. As discussed in the previous section, failure is unlikely to occur in just this way. We believe that most of our conventional structural systems will fail at some connection between the vertical supporting systems and the flexural members. The reinforcement will tear out of the supporting column or walls and the concrete having already sheared through will cause the slab to drop almost in one piece. This form of failure is, of course, undesirable but is likely in many of the systems that exist today.

2.2 DEFINITION OF FAILURE

For the purposes of planning and conducting the program being outlined here, we believe that failure should be considered to be that point where the structural system has deformed to the point that occupancy is no longer possible within the space contained by the system. This failure may be a gradual flexural failure on a specially well-designed system, or it may be a total collapse resulting from weaknesses at points of support or interaction. We believe that in testing models, loading should be continued to the point where the area still remaining under the load-deformation curve can be considered to be insignificant, or until the loading mechanism is no longer operative.

CHAPTER 3

MODELS

3.1 MATERIALS

3.1.1 Concrete. Ma ıy materials have been used to simulate concrete in small scale model studies. Those mentioned by European authorities include gypsum plaster with or without aggregates of sand or asbestos fibers, and micro-concrete making use of sand, cement and water. Aggregates such as pumice, rubber, cork and vermiculite have been used to simulate foundation materials. Heavy aggregates such as barite or litharge have been employed to increase the mass of the micro-concrete for dynamic studies. Mention has also been made in literature of mixes employing various resins with fibrous material as aggregate for special studies.

We believe that most of the research anticipated to be conducted at Fort Belvoir will be best accomplished by employing micro-concrete made with sand, cement and water.

3.1.2 Cement. For most work, type I portland cement should be used unless time becomes a problem, in which case the use of type III cement may be desirable. The cements should be clean, fresh and free of lumps and kept in good dry storage. It may be well to consider blending at least four different brands in order to avoid characteristics which may be unique with a particular brand.

3.1.3 Aggregate. The heads of the various European laboratories with whom we have communicated have suggested that gap grading be employed for designing the micro-concrete mixes. Thus, we suggest that the sand material be fractionated in the following size groups by making use of ASTM standard sieves:

minus 100
50 to 100
30 to 50
16 to 30
8 to 16
4 to 8

This material should be thoroughly dried and stored in separate containers which will keep it dry. The material should be recombined in the amounts determined for a particular mix design.

Desired compressive strengths may be obtained on a trial mix basis with the water cement ratio controlling the strength, and sand cement ratio controlling workability. Rowe states that his group in England has found that sand cement ratios range from 2.5 to 4.0, and that water cement ratios range from 0.4 to 0.55. Within this range they have achieved compressive strengths based on tests of 4 inch cubes which range from 4,000 to 10,000 pounds per square inch in 28 days.

3.1.4 Steel. Many types and sizes of wire are available for use as reinforcement in micro-concrete models. Some of the major steel companies are now producing deformed wire for use in welded wire fabric. We believe that this material will be most useful in concrete model studies. However, smaller wire will be required in many instances than is available from these sources. Wire which is called "black annealed" is good with respect to its basic physical properties and exhibits low yield plateaus. It also has strain hardening characteristics similar to intermediate grade reinforcement. This wire is coated with a black oxide which must be removed. This can be accomplished by pickling in a concentrated solution of hydrochloric acid.

The bond characteristics of a smooth wire must be considered. The best method which we have found for improving bond, and which has been referred to by others, is by inducing an artificial rusting to the wire through exposure to high humidity and air for a few days. We have found that this rusting can become quite extensive without reducing the strength of the wire measurably or changing its physical characteristics. Other methods of improving bond characteristics which have been mentioned and which we have considered are using threaded steel rods and wires which have been scored by pressing between files. Plain wires could also be coated with epoxy just prior to casting the concrete in the model. We do not believe the latter method has been attempted by anyone but suggest it as a possible method of improving bond if other methods do not produce the desired results. Most of the so-called bright wires exhibit strengths and stress-strain curves considerably different than those found in intermediate grade reinforcement. Some of these wires can be annealed to produce characteristics which approximate those of prototype reinforcement. This should be accomplished on a trial and error basis, and a range of sizes should finally be selected and stored in quantity. A particular reinforcement may then be selected which gives the characteristics desired to correlate the model behavior with that of a prototype if desired.

3.2 CALIBRATION AND TESTING OF MATERIALS AND INSTRUMENTATION

3.2.1 Concrete. In order to correlate the strength and behavior of small scale reinforced mortar models, it is necessary that the material properties in the model be in some way related to properties measured using standard test specimens. These properties must in turn be related to those of the materials in the prototype.

Studies conducted at M.I.T. have indicated that attempts to scale compression test cylinders have produced erratic and inconsistent results. None of the Europeans who were interviewed referred to this problem. We conclude that either this problem has been solved in the laboratories in Europe or no effort has ever been made to work with very small test samples. All investigators indicated good correlation between model results and test specimens made in sizes consistent with those which are

usually made for mortar studies. We believe that a standard 2" x 4" mortar cylinder should yield acceptable results for establishing concrete properties from standard testing. However, because the question has been raised, we suggest that a modest investigation be conducted whereby rectangular beams are tested to evaluate the results from cylinder tests on the basis of the known flexural characteristics of reinforced concrete. For example, over-reinforced model beams could be made in several scales and tested with third-point loading over a simple span. These beams could correspond to well-documented laboratory tests of larger scale beams which have been reported in the literature (47A). The compressive strength of concrete may be thus determined by making use of the material developed by Hognestad, Hanson and McHenry (48A), in which parameters are used to correlate the compressive stress block in a flexural member at ultimate load with results obtained from compressive tests on cylinders. The values of the parameter K_3 have been determined so that results from tests on 6" x 12" cylinders may be used to evaluate the compressive stress block. Europeans use cubes as compression test specimens and use modified values for K_3 accordingly. It may be necessary to adjust this quantity for correlation of small scale model results and compressive test specimen results.

These same tests may be used to evaluate the modulus of elasticity of concrete by measuring deflections prior to flexural cracking. The values of modulus of elasticity thus obtained may be compared with those determined from the 2" x 4" test cylinders.

The 2" x 4" cylinders may be used for split cylinder tests for establishing the tensile strength of the micro-concrete. If necessary, creep and shrinkage studies should be made on specimens which correspond in configuration to prototype creep and shrinkage specimens.

3.2.2 Steel. Tensile tests should be conducted on model reinforcement to establish the stress-strain characteristics. If these characteristics are unsatisfactory, it may be possible to improve the relationship by annealing the steel. Further tensile tests should then be conducted to establish the revised characteristics.

Bond characteristics of the reinforcement may be evaluated prior to incorporating the steel in a model by making pull-out tests in which the specimens correspond in scale to those reported for conventional reinforcement (49A) and (50A). However, we believe a more effective evaluation may be accomplished by again resorting to rectangular model beams in various scales. These should be under-reinforced and tested with third-point loading over a simple span. The behavior of these model beams throughout the entire range of loading will provide one means for comparison with the behavior of prototype test beams and slabs in all respects. The bond characteristics may be evaluated by comparing the crack patterns which develop with those of the corresponding prototype beams. The load-deflection characteristics of the beams will provide a measure of the stress-strain characteristics of the reinforcement as they affect the

behavior of the reinforced test members.

The shear strength of mortar models could also be evaluated with the use of beam specimens. Punching shear models might also be built and compared with Elstner's full scale studies (41A).

3.2.3 Scale. From the standpoint of test results all would agree that the ideal scale is 1 to 1. We could be certain of our test results if the prototype itself were tested. Obviously, this in an impractical, costly method of evaluating structural performance. On the other hand, while the cost of model materials, testing equipment and space requirements are reduced as the size of the model is reduced, it has been found not inexpensive to produce and test very small scale micro-concrete models. Furthermore, many feel that the precision of results diminishes as the scale becomes smaller. It should be mentioned, however, that with proper care, good correlation has been achieved at very small scales. Nine-panel flat plate (slab) configurations have been scaled from 1.33 size down to 28 with essentially identical results. Let us define scale factor as the ratio of prototype dimensions to model dimensions. For example, a scale factor of 5 denotes that the size of the prototype is five times that of the model in linear dimensions. Considering all model design aspects which should include availability of reinforcement sizes, difficulty in handling wire reinforcement, placing concrete, test space requirements, instrumentation, and loading limitations, the scale factors which have been selected generally for mortar models by the European model analysts have ranged from 1 to 16. Apparently most researchers favor a scale factor of between 8 and 16.

Usually, the available size of reinforcement and the relationship that the size of this steel has to prototype bar size will constitute the first step in selecting a scale factor. Another consideration that discourages the use of very small scales is the extreme fragility of columns and beams cast with the slab. As noted by University of Illinois investigators, it is extremely difficult to strip these tiny models without breaking parts. Other considerations which must be taken into account in deciding on a scale factor will be forming difficulties, placement of concrete, fabrication of reinforcement cages, method of loading, and complexity of configuration.

A very important consideration is that the total magnitude of load increases with the square of the scale factor. For example, a model made to a scale of 5 would require 4 times as much total load as a model with a scale factor of 10. As a general rule, however, the scale factor should be kept as small as practicality will permit.

3.3 LOADING

Many systems of loading models have been employed by investigators. These are hydraulic jacks, dead weight systems, air and fluid bag systems, and atmospheric pressure or vacuum systems. Each is discussed below and

the advantages and disadvantages as we see them are presented.

3.3.1 Hydraulic Jack Arrangements.

Advantages

With a properly designed framework, the loads may be located in a variety of patterns. It is relatively easy to control the direction of load application. Control of the magnitude of loading can be done with adequate precision. The application of load and unloading can be accomplished rapidly and easily with a properly designed reaction framework around the model.

Disadvantages

The framework and hydraulic equipment is more costly than that required for other loading methods. It is difficult to maintain a load at the advanced stages of loading. Local failures or spontaneous distortions result in a temporary reduction in hydraulic load. Hydraulic jacks must apply their load at discreet points. However, this disadvantage can be partially alleviated by making use of load-distributing pads. In pull rod systems, the holes in the slab can interfere with the fracture patterns. Unless the loading system is carefully designed, it may be difficult to get a good view of the crack patterns as they develop during loading.

3.3.2 Dead Weight Systems.

Advantages

The load is maintained throughout testing regardless of failures or deformations until the model is incapable of sustaining the load. Extreme accuracy is possible as the dead weight suspended from the model may be carefully weighed before or after testing.

Disadvantages

Again, the load is applied through discreet points by tension rods. It is difficult to position the loads indiscriminately and maintain balance through the suspension system supporting the dead weight. Loading and unloading is cumbersome, although hydraulic jacks can be used to lift and lower platforms on to which the loads are placed. If the scale of the model is large, a large magnitude of weight may be required. Thus, dead weights can be quite awkward and perhaps costly. The appearance of a testing arrangement involving dead load is not as pleasing as those which depend upon hydraulic pressure or the other more sophisticated methods of load application. The

crack interference and viewing problems mentioned for the jacking system prevail here also.

3.3.3 Air or Fluid Bag Type Loading.

Advantages

This method of loading can be controlled quite positively and is a good method of applying uniform load. It has the advantage over the atmospheric or vacuum type loading method in that greater unit pressures are possible. The load can be maintained with a fair degree of certainty as local failures and spontaneous deformations take place.

Disadvantages

It is difficult to observe the loaded surface of a model as testing takes place. Rather cumbersome and heavy framework is necessary to contain the air bag to develop the load against the model. It may be difficult to control the loading at areas of extreme change in shape.

3.3.4 Vacuum Technique.

Advantages

The top surface may be viewed throughout the loading process by using materials for an air seal membrane which are transparent. The loading is uniform on flat surfaces. The equipment necessary to accomplish this loading is relatively inexpensive. The size of the model is not particularly significant to limiting the total magnitude of load. The major consideration here is that the larger the area to be loaded, the more desirable it is to have a large surge tank. For most studies, atmospheric pressure will give a sufficient degree of load.

Disadvantages

It is difficult to observe the side of the model opposite that which is receiving load. However, we believe this can be overcome and will present insignificant problems. We will cover this matter in our report which will be prepared for Phase 2 of this program. Obviously the magnitude of loading is limited by the prevailing atmospheric pressure. The loads are always normal to the surfaces against which the atmospheric pressure is acting. This is not a disadvantage if the surfaces are flat but may be considered so for shell structures or other structures on which the surfaces are curved. There is a possibility that some error may be introduced due to the reinforcing action of the membrane which is used to seal against the flow of air through the model. Rowe has indicated that he has solved this problem by introducing a lubricating medium to reduce the friction between the membrane and the loaded surface. We believe that

a well-equipped model testing facility should include provisions for hydraulic jack loading and vacuum loading.

CHAPTER 4

CONCLUSIONS - PART I

A great deal of work has been done with small scale models of buildings and building components. However, most of it has been confined to studies of response to load within the elastic range. European effort on the study of inelastic response in reinforced concrete has exceeded that in the United States to a considerable degree. Nevertheless, the total experience in small scale models constructed with micro-concrete is rather meager. On the other hand, most of the research which has served to further our knowledge of the behavior of reinforced concrete has been conducted by testing specimens which could be considered as models, although not small scale. Thus, if one has faith in the reliability of the work done in the laboratory which has founded much of our design procedures and building code development, he endorses model studies. The only question to be resolved then becomes that of scale. We quote R. E. Rowe here who states:

> "When model tests are discussed, the question invariably arises: 'What is the correlation between results obtained from a small-scale model and the actual behavior of the structure?' I am tempted in replying to this question to quote Hossdorf, who said that the question is about as meaningful as asking a designer to define the correlation between the results obtained by a classical method of analysis and the actual behaviour of the structure."

We firmly believe that micro-concrete models scaled from 8 to 16 can be made and tested to represent prototype behavior more accurately than any known analytical methods. In certain cases it may be necessary to resort to smaller scale. This is especially true with respect to interaction between reinforced concrete structural elements assembled together to form complete roof or floor systems. However, as with any engineering research, skill and engineering judgment are necessary for accurate and valuable results.

Considerable testing has been done on floors and roofs of full scale structures but most of it may be called "proof testing". Thus, the purpose has been to establish the safety of the structure without damaging it. Very few instances have been reported of tests conducted which produced failure in full scale roof or floor systems.

We also believe that knowledge of the strength and behavior of interacting reinforced concrete structural elements under loads producing large deformations is very limited. A well conceived and executed program involving the study of small scale reinforced concrete models will greatly enhance this deficiency of knowledge.

PART II

CHAPTER 5

LONG RANGE PROGRAM

While a long range program developed in detail may be desirable from the viewpoint of financial and personnel budgeting, experience in research has taught us that changes in planning are required as we learn during the course of the work. The long range objective, i.e., evaluation of the ultimate load-carrying capacity of existing reinforced concrete structures is, of course, very ambitious. Therefore, in order to glean as much as possible from a limited amount of work, the program must be kept fluid and built around a carefully considered step-by-step development. However, the fundamental objective must serve as the basis for the program regardless of how it develops.

5.1 TWO-PHASE PROGRAM

We recommend that two phases be planned initially in detail and that the results obtained therefrom be employed in the planning of subsequent phases. Results obtained by others must be considered as work progresses and additional phases are programed.

5.1.1 Phase 1. The first phase is recommended to serve a dual purpose. This phase involves tests conducted on a series of reinforced concrete elements duplicating in small scale work on full scale members which have been performed by experienced researchers. Their effort is very well documented and has served importantly in the development of our understanding of the fundamental behavior of reinforced concrete.

The first purpose of this study is to establish that reinforced concrete elements of the size to be tested behave essentially under load as similar larger members. If minor differences do become apparent, it is probably that correction or distortion factors can be established so that they may be used in evaluating the performance of the three-dimensional structures tested in subsequent studies. We do not anticipate that significant distortions will be observed.

The second purpose of Phase 1 is to develop and evaluate model fabrication, instrumentation and testing techniques. In addition, the relationship between the properties of concrete, as established from tests on 2" x 4" cylinders and the corresponding properties as they affect the performance of beams, columns and slabs, may be determined.

Phase 1 then, as recommended, consists of five series of tests, each patterned after a prototype series reported in the technical literature. The first of these series includes six beams termed "flexural beams" selected from the members tested at the Portland Cement Association

laboratories and reported by Dr. Eivind Hognestad in PCA Development Bulletin D53. The details of suggested members are presented in Figures 5.1 through 5.6. The second series consists of eleven members called "bond beams" selected from the same series of tests reported in Bulletin D53. These members are illustrated in Figures 5.7 through 5.17. The third series was also selected from work done on slab shear at PCA and consists of four test members shown in Figures 5.18 through 5.21. This work is reported in PCA Development Department Bulletin D47. The fourth series termed "shear beams" is modeled after selected members included in the work reported by Moody, Viest, Elstner and Hognestad in the ACI Journal in December, 1954. These are shown in Figures 5.22 through 5.24. The last series is comprised of the four model test columns shown in Figure 5.25. The original work was done by Hognestad and is reported in University of Illinois Engineering Experimental Station Bulletin 399.

5.1.2 Phase 2. For Phase 2, the three-dimensional slab systems shown in Figures 5.26 through 5.22 are recommended. These proposed floor or roof slabs correspond to the systems presented in the first part of this report as those which have been employed most commonly in this country in reinforced concrete construction. The spans and other dimensions which are shown were selected as a result of our interrogation of a number of consulting structural engineers who have had many years of experience in the design of such structures.

We suggest that three test specimens of each type shown be cast at a scale of about 16. The dimensions should remain constant for each set of three but the reinforcement percentage should be varied to correspond to the requirements for normal roof loading, medium floor loading (about 200 psf).

5.2 TIME/MANPOWER REQUIREMENTS FOR PHASES

We estimate that the following time and manpower requirements, including analysis and report, will be necessary for Phases 1 and 2:

Work	Time	Manpower
Phase 1	6 months	1 engineer 1 technician
Phase 2	3 years	2 engineers 2 technicians

At or near the conclusion of Phase 2, the results thus far obtained will serve as an important basis for the development of further work.

Such things as multiple floors, lateral loading, and foundation conditions should be considered for these future phases. We repeat here, however, that these future phases can be most effectively planned after evaluation of the results of Phase 2.

5.3 MODEL SCALE, MATERIALS AND INSTRUMENTATION

5.3.1 Scale. As discussed previously, the micro-concrete models proposed for study should be built with scale factors from 8 to 16. It may be necessary to consider smaller scales (larger scale factors) in certain instances because of space or loading requirements, but the scale should not be reduced beyond 16 unless absolutely necessary.

5.3.2 Materials. The following micro-concrete mix is recommended as a standard:

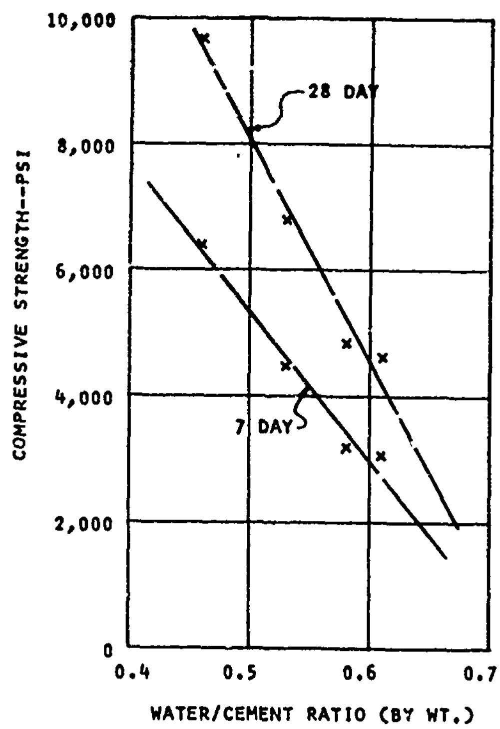
Aggregate Grading

Size No.	Indiv. Ret.	Cum.
8	32%	32%
16	16%	48%
30	16%	64%
50	7%	71%
100	5%	76%
Pan	12%	88%
#80 Silica Flour (Tamms)	12%	100%

Mix Proportions

	% of Total Dry Materials
Aggregate	84
Cement	16

Water as required from w/c - strength chart. The quantities shown are based on the aggregates being bone dry. Strength variation may be achieved by varying the water-cement ratio as shown in Figure 5.1a.



2 X 4 IN. CYLINDER COMPRESSIVE STRENGTH OF GRADED MICRO-CONCRETE

Figure 5.1a

Wire reinforcement must be acquired and tested so that the stress-strain characteristics approximate those of prototype reinforcements. The following assortment of wire should be accumulated:

Type Gage	Gage	Prototype to Scale Factor of 16
U.S. Standard	27	# 3
U.S. Standard	24	# 4
U.S. Standard	21	# 5
U.S. Standard	19	# 6
U.S. Standard	18	# 7
U.S. Standard	17	# 8
U.S. Steel	16	# 9
U.S. Steel	15	#10
U.S. Standard	13	#11
U.S. Steel	12	#14
U.S. Steel	9	#18

An attempt should be made to achieve stress-strain characteristics typical of ASTM A15 Intermediate grade, ASTM A15 Hard grade, ASTM 432, and A431 for each of the above wire sizes.

All wire reinforcement should be given a coating of rust by exposure to water and air in a shallow tank made of Plexiglas. Any mill scale should have been previously removed by pickling in a concentrated solution of hydro-chloric acid. The same tank may be used for both purposes. The suggested arrangement for testing the steel is shown in Figure 5.39.

The method illustrated in Figure 5.39 may also be used to calibrate wire which has been instrumented with SR-4 strain gages. This procedure is suggested because it will be very difficult to apply strain gages to small wires and depend on the conversion of measured strains to stress by resorting to the modulus of elasticity of the steel. Therefore, a stress-strain relationship should be obtained for each instrumented wire on the basis of test.

5.3.3 Instrumentation. For the present we recommend that the instrumentation equipment and supplies be restricted to the following:

1. SR-4 instrumentation for model strain and force measurements.

   a. Assortment of SR-4 gages
   b. Wire and soldering tools
   c. SR-4 read-out instrument
   d. Recording oscillograph (4 channels, minimum)

2. Assortment of dial indicators reading to 0.001 inch (8, minimum).

3. Surveyors level -

   a. Assortment of steel machinists scales marked in 0.01 inch.

4. Water and mercury manometers for pressure measurements.

5. Compressometer for measuring stress-strain characteristics of 2" x 4" micro-concrete cylinders.

5.4 MODEL LOADING

5.4.1 Live Load. We believe that most of the vertical loading and some horizontal loading may be best accomplished with vacuum techniques, but there may be occasions when it is desirable to use hydraulic jacks for loading. The test table shown in Figure 5.35 has been designed to accommodate either method of load application and it can be used for vertical or lateral loading. Figure 5.40 shows a schematic vacuum load frame for the test table.

Item	Source	Est. Cost
11. Concrete Scales*	Soiltest L-855	$ 112.00
12. Drill Press*	Sears Roebuck & Co.	110.00
13. Band Saw*	Sears Roebuck & Co.	125.00
14. Table Saw*	Sears Roebuck & Co.	170.00
15. Radial Saw*	Sears Roebuck & Co.	250.00
16. Jointer*	Sears Roebuck & Co.	85.00
17. Sander*	Sears Roebuck & Co.	60.00
18. Assorted SR-4 Gages*	Baldwin-Lima-Hamilton	500.00
19. Strain Indicator*	Baldwin-Lima-Hamilton	600.00
20. Recording Oscillograph and Amplifiers	Sanborn	1,000.00 per channel to 6 channels 600.00 per channel from 7 to 14 channels
21. Dial Gages*	Soiltest LC-8	210.00
22. Test Table*	To be built	1,000.00
23. Work Benches*	To be built	500.00
24. Storage Racks*	To be built	250.00
25. Vacuum Pump	Welsh 1402B	350.00
26. 55-Gal. Drum (2) Surge Tanks	To be modified	60.00
27. Pipes, Valves, Hose and Fittings		100.00
28. Hydraulic Pump	Simms Engineering	850.00
29. Hydraulic Control Panel	Simms Engineering	600.00
30. Hoses and Fittings	Simms Engineering	750.00

*Required for Phase 1.

	Item	Source	Est. Cost
31.	Jacks and Ball Joint	Simms Engineering	$ 210.00 each
32.	Capping Table Hood*	To be built	200.00
33.	Miscellaneous Hand Tools*		100.00
34.	Precision Level*	Gov't Surplus	300.00
35.	Machinists Rules (10)*		25.00
36.	Manometer (Mercury)	Central Scientific 94125	70.00
37.	Unistrut Framing		100.00
38.	Switchbox*	Shall cross	150.00
39.	Plexiglass as required for framing including ethylene dichloride for cement		300.00
40.	Teflon sheet (0.003 thick)		150.00
41.	Polyethylene sheet (0.010 thick)		50.00
42.	Assorted plastic coated plywood		100.00
43.	Misc. structural steel small angles, channels and I beams		200.00

*Required for Phase 1.

5.7 ESTIMATED TIME REQUIREMENTS FOR EACH MODEL IN PHASE 2

1. Design

   a. Form design and drawings
   b. Instrumentation design
   c. Reinforcing design and drawings
   d. Load frame design

Engineer days	8
Draftsman days	4

2. Model Fabrication

   a. Mold fabrication
   b. Reinforcement cage assembly

Engineer days	2
Technical days	10

3. Instrumentation of steel and calibration

Engineer days	1
Technician days	2

4. Model assembly

   a. Steel placement
   b. Casting model and test samples

Engineer days	1
Technician days	2

5. Curing

Days	14

6. Assembly in test frame and Instrumentation

Engineer days	2
Technician days	8

7. Test

Engineer days	2
Technician days	4

8. Data reduction, analysis and report

Engineer days	25
Draftsman days	10

Total Engineer days	31
Total Technician days	26
Total Draftsman days	14

Suggested lost time multiplier = 1.25

Wiss, Janney, Elstner & Associates 870 Northwest Highway • Des Plaines, Illinois	O. C. E - O. C. D. MICRO - CONCRETE MODEL STUDIES	SHEET NUMBER
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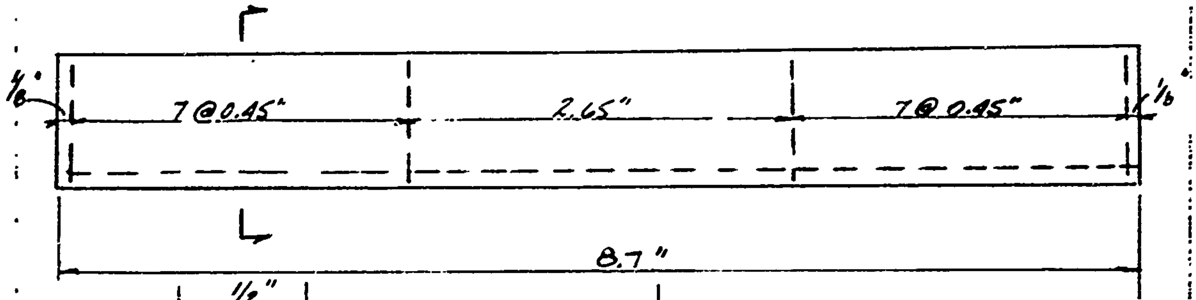
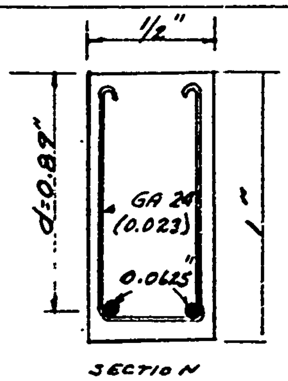
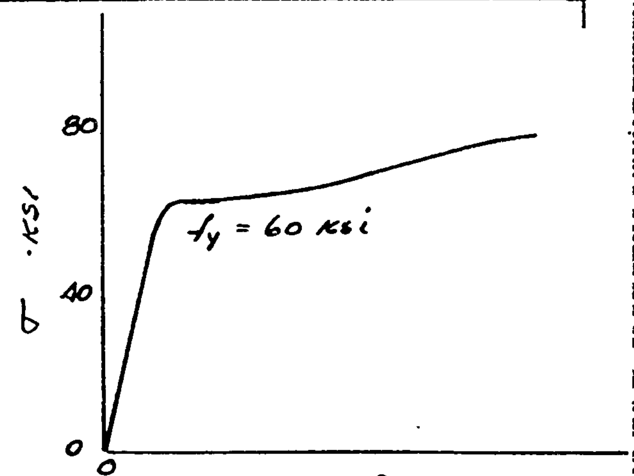
FLEXURE TESTS



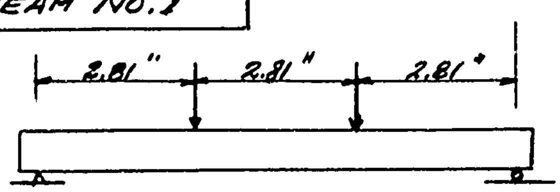




REINFORCEMENT STRESS-STRAIN CURVE

SPECIFICATIONS

f_c' = 4500 PSI
AGE AT TEST – 14 DAYS
RUSTED WIRE REINF.
MAKE 10 2" x 4" CYL.
SCALE: 16
PROTOTYPE: PCA BUL D-53 BEAM NO.1



TEST SET-UP

Figure 5.1

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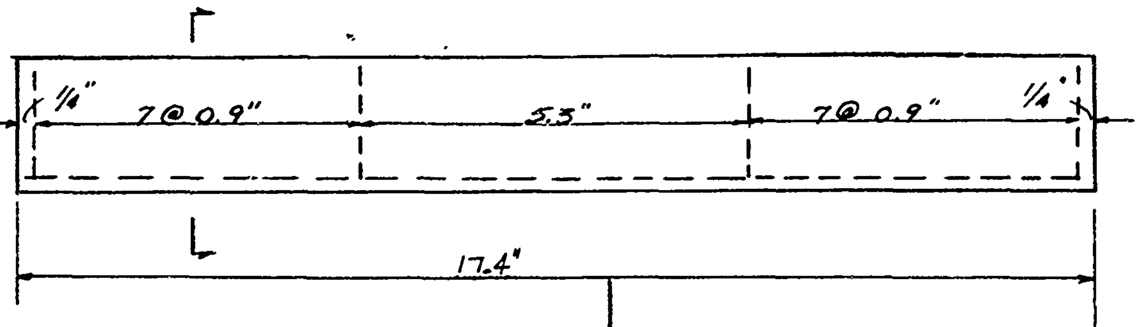
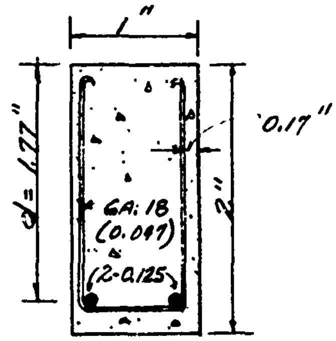
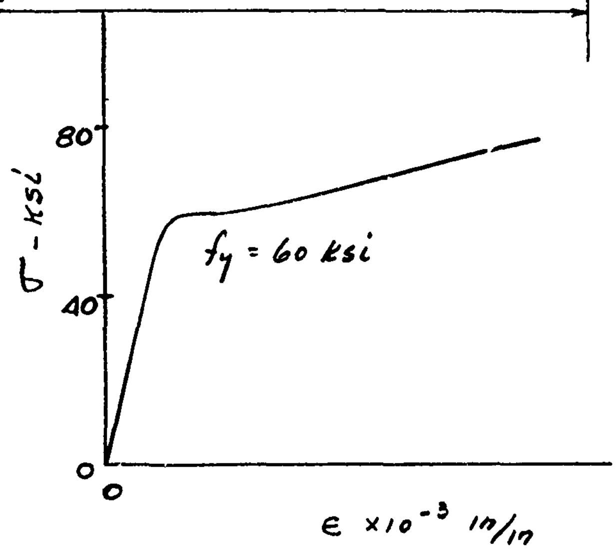




SECTION



REINFORCEMENT STRESS-STRAIN CURVE

SPECIFICATIONS

f_c' = 4500 PSI

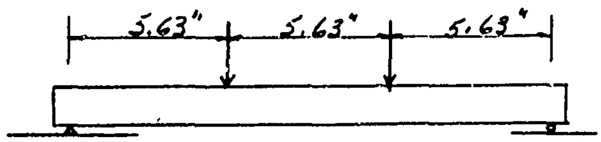
AGE AT TEST 14 DAYS

RUSTED WIRE REINF.

MAKE 10 2"x4" CYL.

SCALE FACTOR: 8

PROTOTYPE: PCA BUL. D-53 BEAM NO 1



TEST SET-UP

Figure 5.2

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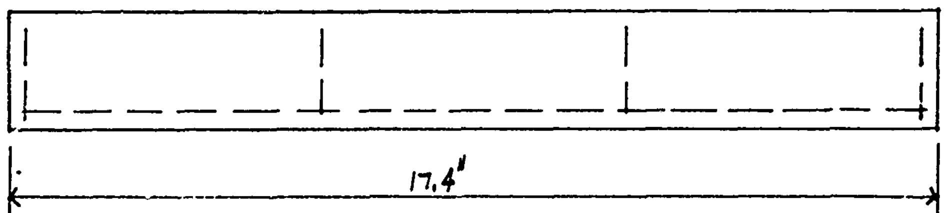
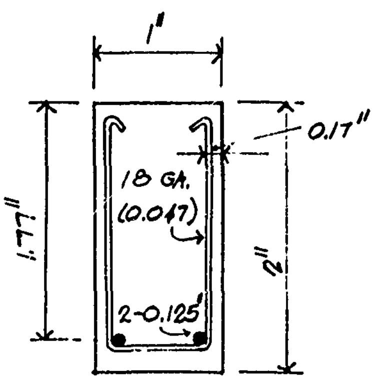
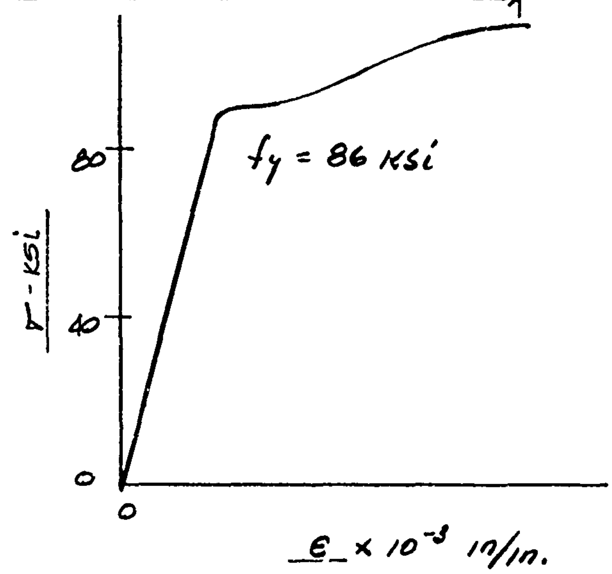
FLEXURE TESTS





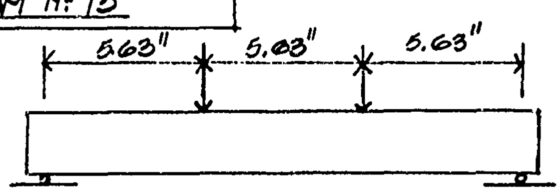
SECTION SPECIFICATIONS



REINFORCEMENT STRESS-STRAIN CURVE

f'_c = 7000 psi
AGE AT TEST - 14 DAYS

MAKE 10 - 2" × 4" CYL.
SCALE FACTOR: 8
PROTOTYPE: PCA BUL. D-53
BEAM No. 13



TEST SET-UP

Figure 5.3

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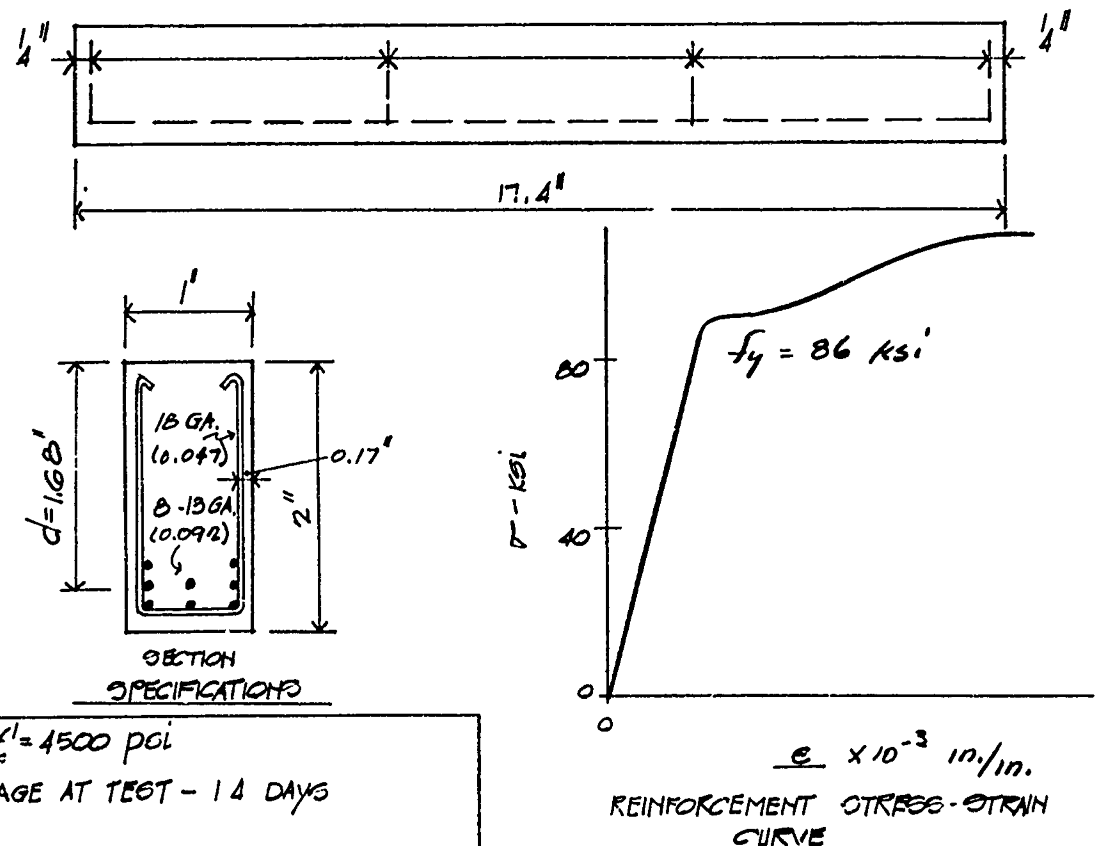
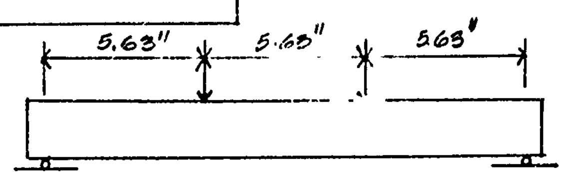
FLEXURE TESTS



f'_c = 4500 psi
AGE AT TEST - 14 DAYS

MAKE 10-2"x4" CYL.
SCALE FACTOR: 8
PROTOTYPE: PCA BUL. D-53
BEAM No. 12



TEST SET-UP

Figure 5.4

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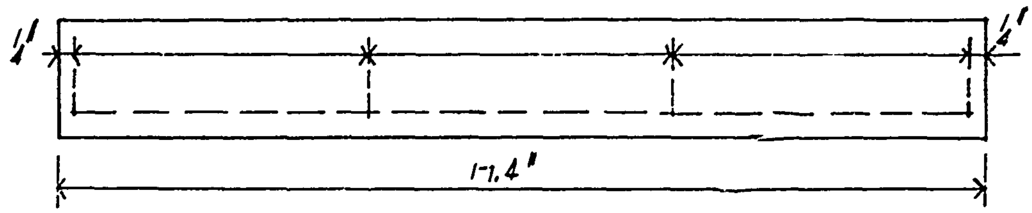
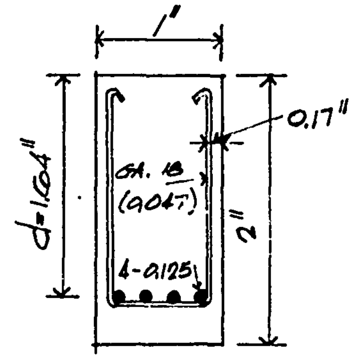
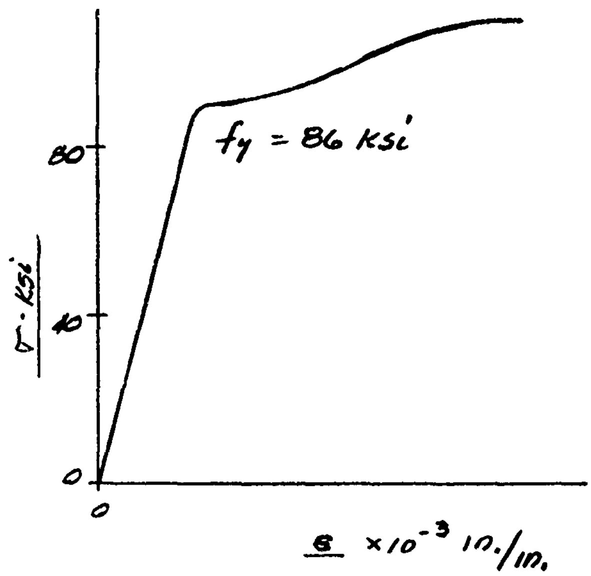
FLEXURE TESTS





SECTION SPECIFICATIONS



REINFORCEMENT STRESS-STRAIN CURVE

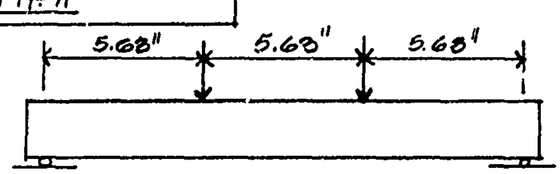
f'_c = 3700 psi

AGE AT TEST - 14 DAYS

MAKE 10 - 2"x4" CYL.

SCALE FACTOR : 8

PROTOTYPE : PCA BUL. D-53 BEAM No 11

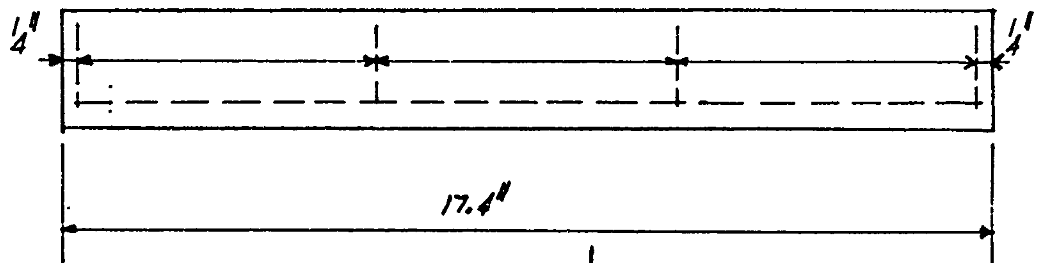


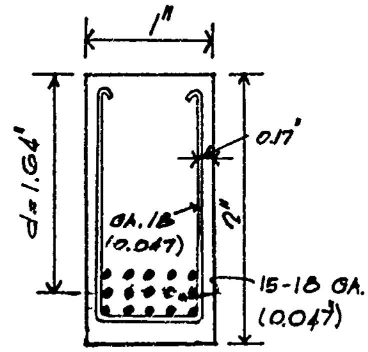
TEST SET-UP

Figure 5.5

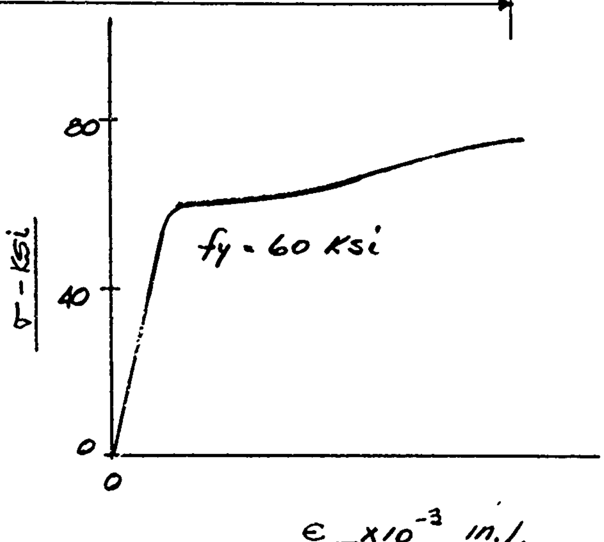
Wiss, Janney, Elstner & Associates 570 Northwest Highway • Des Plaines, Illinois	O.C.E. - O.C.D.	SHEET NUMBER
MADE BY H.G · DATE 5/27/65 · CHECKED BY	MICRO-CONCRETE MODEL STUDIES	JOB NUMBER 3385

FLEXURE TESTS





SECTION



REINFORCEMENT STRESS-STRAIN CURVE

SPECIFICATIONS

f'_c = 4000 psi

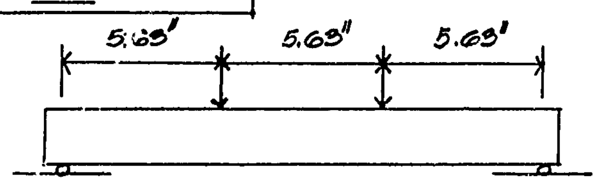
AGE AT TEST — 14 DAYS

RUSTED WIRE REINF.

MAKE 10 - 2"x4" CYL.

SCALE FACTOR : 8

PROTOTYPE : PCA BUL. D-53
BEAM No 4



TEST SET-UP

Figure 5.6

Wiss, Janney, Elstner & Associates 570 Northwest Highway • Des Plaines, Illinois	O.C.E. -O.C.D MICRO-CONCRETE MODEL STUDIES	SHEET NUMBER
MADE BY: HLS · DATE: 5/29/68 · CHECKED BY:		JOB NUMBER

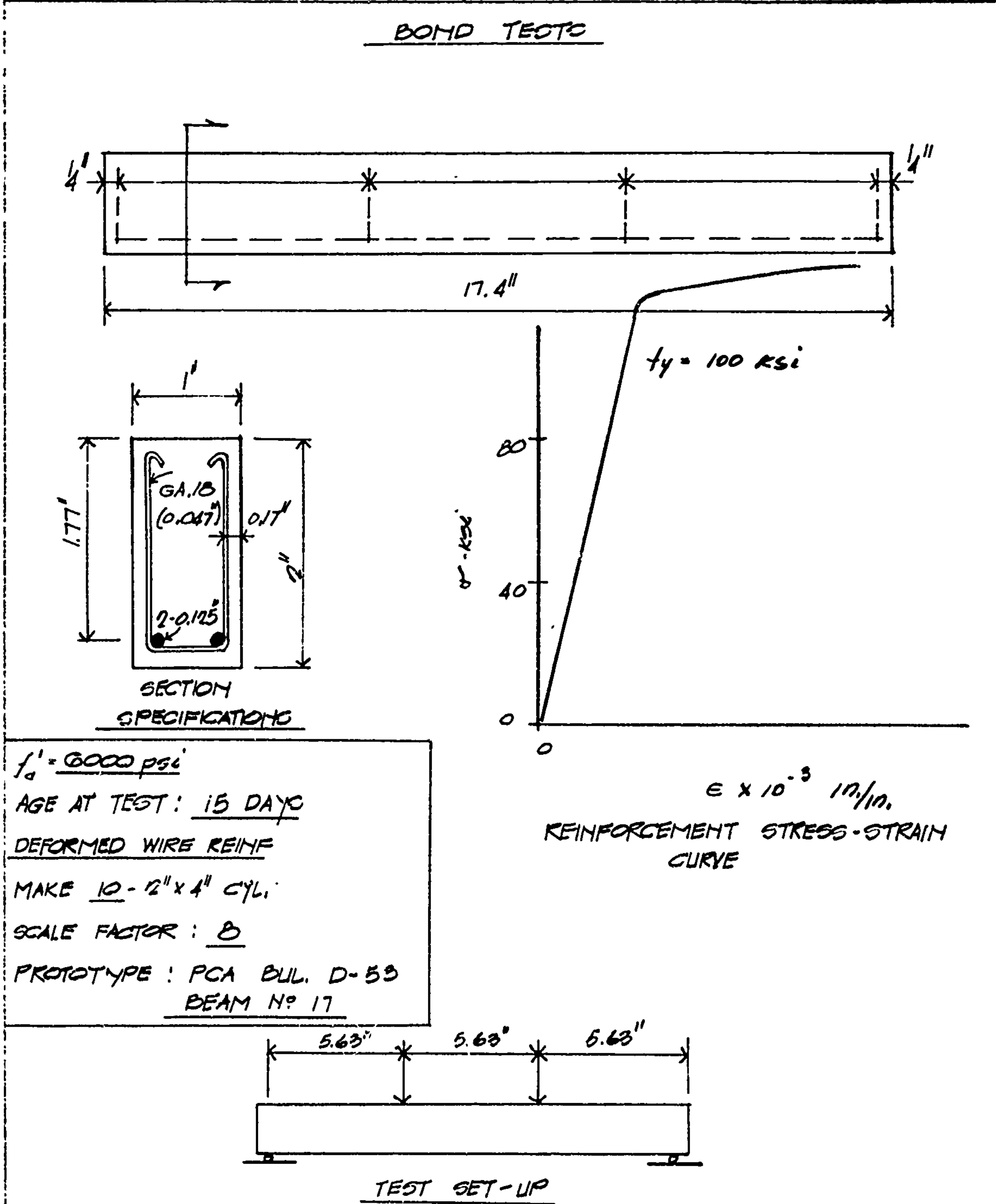
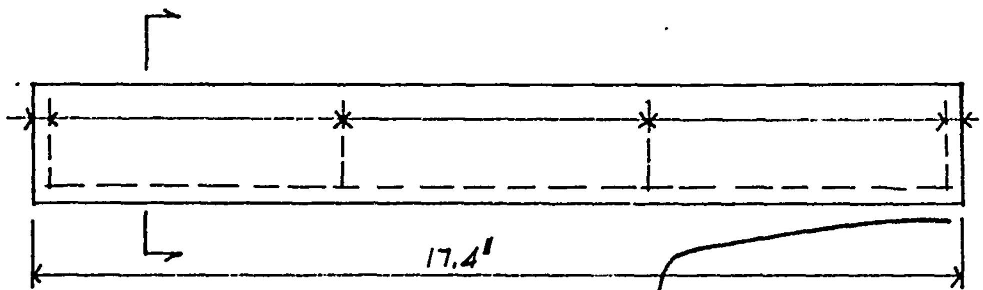


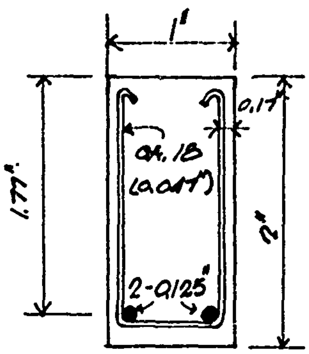
Figure 5.7

1000 H 10

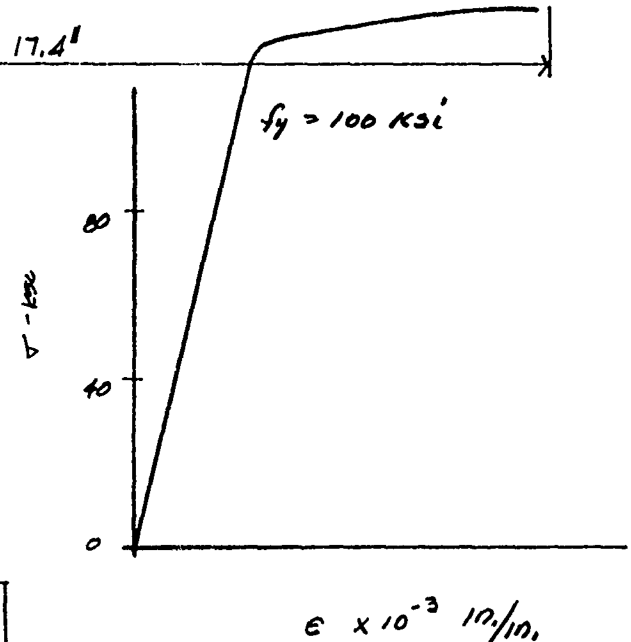
Wiss, Janney, Elstner & Associates 570 Northwest Highway • Des Plaines, Illinois	O.C.E. - O.C.D. MICRO-CONCRETE MODEL STUDIES	SHEET NUMBER
MADE BY: HLS — DATE: 5/29/65 — CHECKED BY:		JOB NUMBER: 3385

BOND TESTS





SECTION SPECIFICATIONS



REINFORCEMENT STRESS-STRAIN CURVE

f'_c = 4500 psi

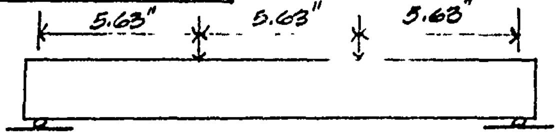
AGE AT TEST : 16 DAYS

DEFORMED WIRE REINF.

MAKE 10 - 2" x 4" CYLS.

SCALE FACTOR : 8

PROTOTYPE : PCA BUL. D-53
BEAM No 17



TEST SET-UP

Figure 5.8

Wiss, Janney, Elstner & Associates 670 Northwest Highway • Des Plaines, Illinois			O. C. E. O. C. D. MICRO-CONCRETE MODEL STUDIES	SHEET NUMBER
MADE BY JRJ	DATE 5-21-65	CHECKED BY		JOB NUMBER 3385

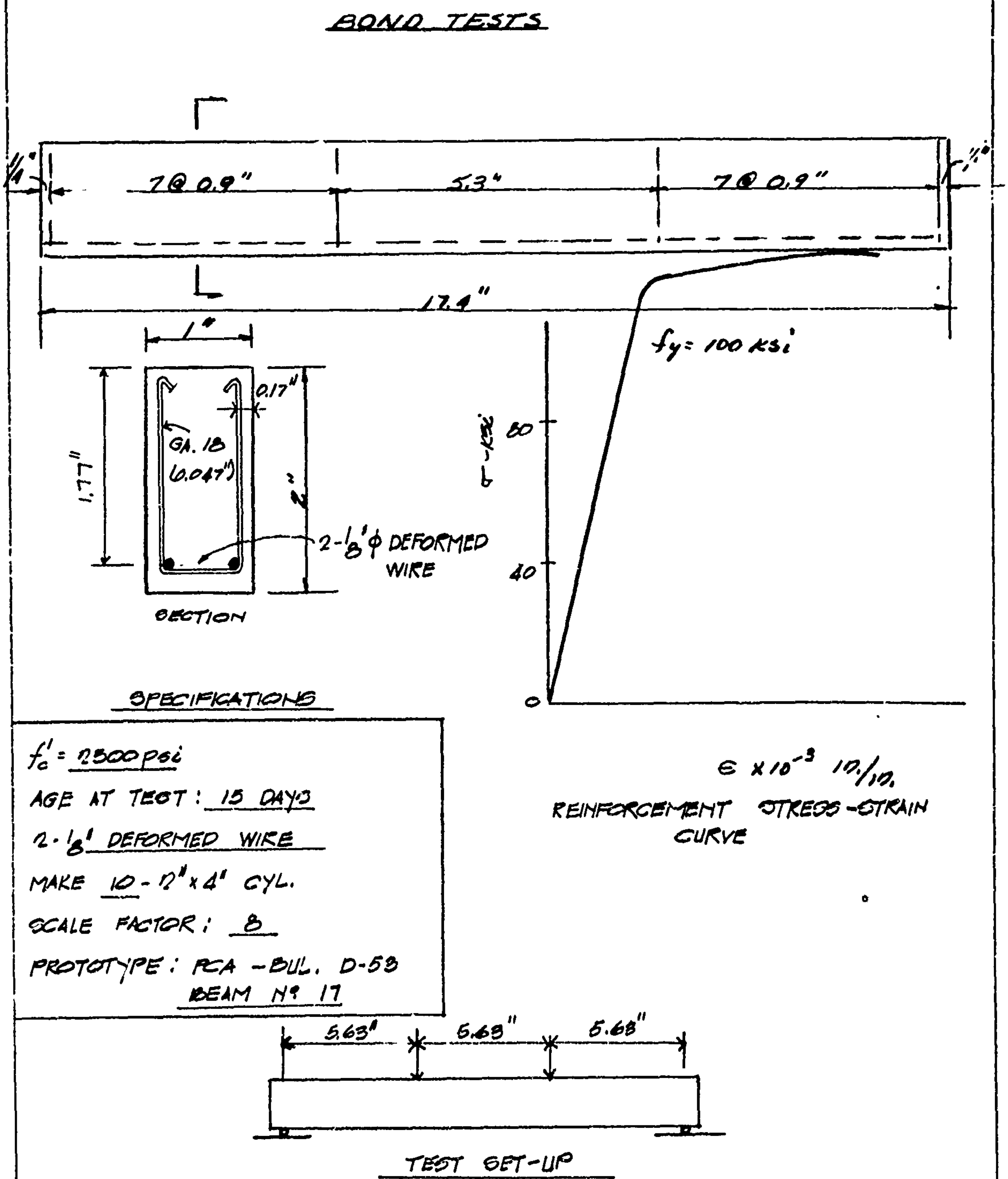


Figure 5.9

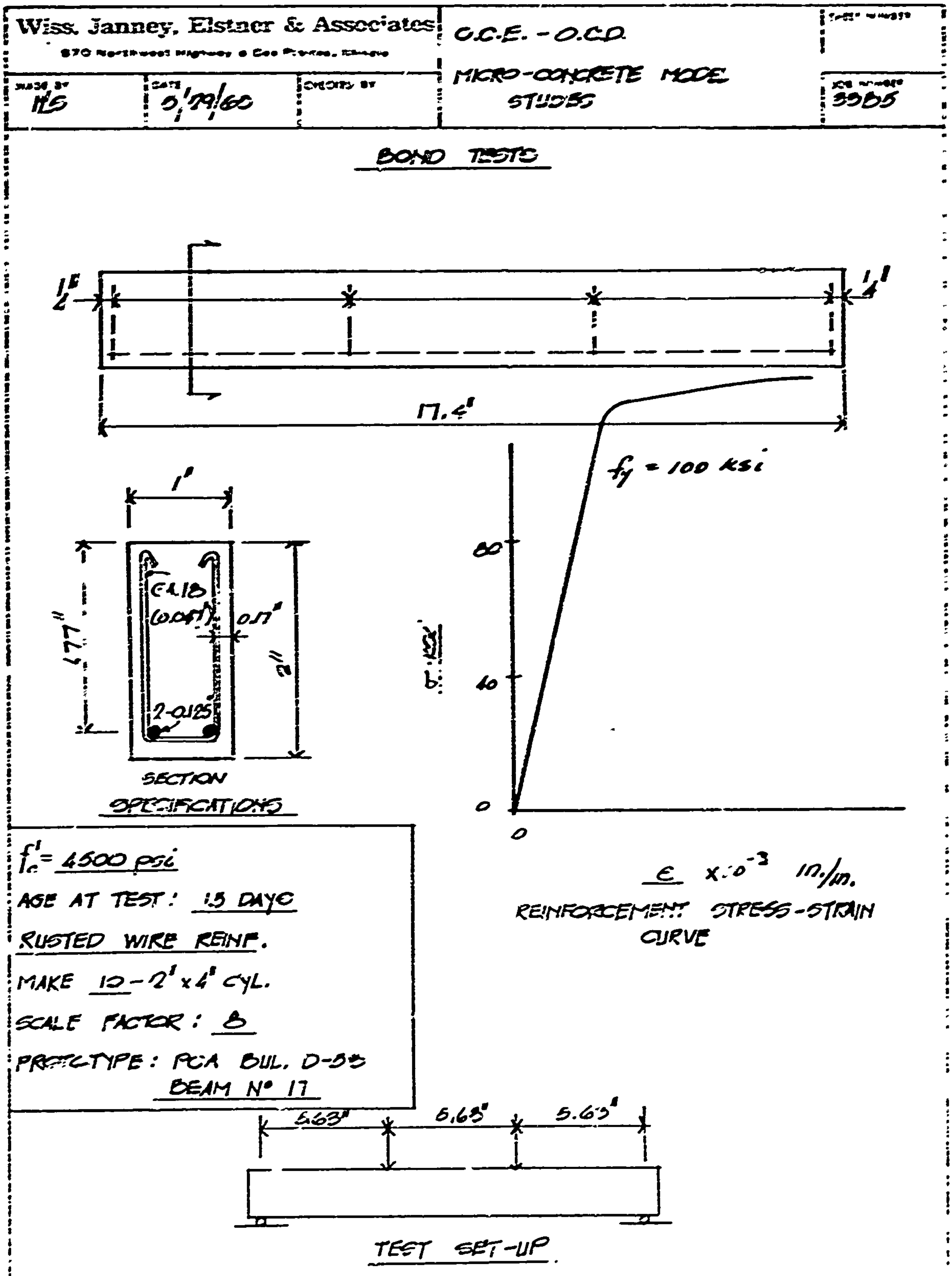
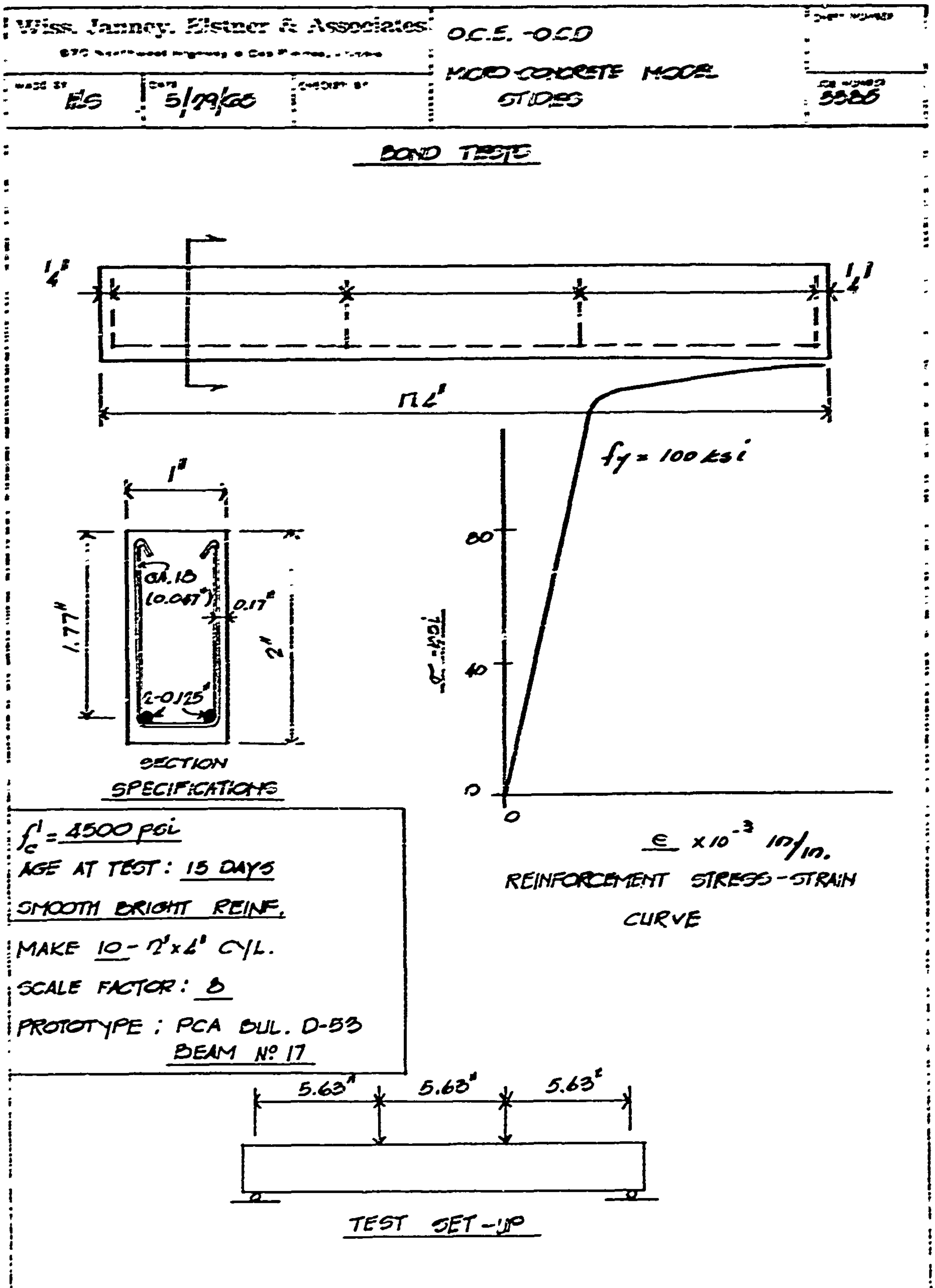

Wiss, Janney, Elstner & Associates
O.C.E. - O.C.D.
MICRO-CONCRETE MODEL STUDIES
MADE BY HS
DATE 5/29/60
CHECKED BY
JOB NUMBER 3365
BOND TESTS
17.4"
1"
.477"
2"
0.17"
2-0.125
SECTION
SPECIFICATIONS
f'c = 4500 psi
AGE AT TEST: 15 DAYS
RUSTED WIRE REINF.
MAKE 10 - 2" x 4" CYL.
SCALE FACTOR: 8
PROTOTYPE: PCA BUL. D-33 BEAM Nº 17
fy = 100 ksi
80
40
0
ε x10^-3 in./in.
REINFORCEMENT STRESS-STRAIN CURVE
5.63"
5.63"
5.63"
TEST SET-UP

Figure 5.10



Wiss, Janney, Elstner & Associates
O.C.E. -OCD
MICRO-CONCRETE MODEL
STUDIES
ES
5/29/66
BOND TESTS
f_y = 100 ksi
80
40
0
σ - ksi
ε x10⁻³ in/in.
REINFORCEMENT STRESS-STRAIN
CURVE
SECTION
SPECIFICATIONS
f'_c = 4500 psi
AGE AT TEST : 15 DAYS
SMOOTH BRIGHT REINF.
SCALE FACTOR : 8
PROTOTYPE : PCA BUL. D-53
BEAM Nº 17
5.63"
5.63"
5.63"
TEST SET-UP

Figure 5.11

Wiss, Janney, Elstner & Associates | O.C.E. - O.C.D. MICRO-CONCRETE MODEL STUDIES | JOB NUMBER 3385
MADE BY HS | DATE 5/21/65 | CHECKED BY

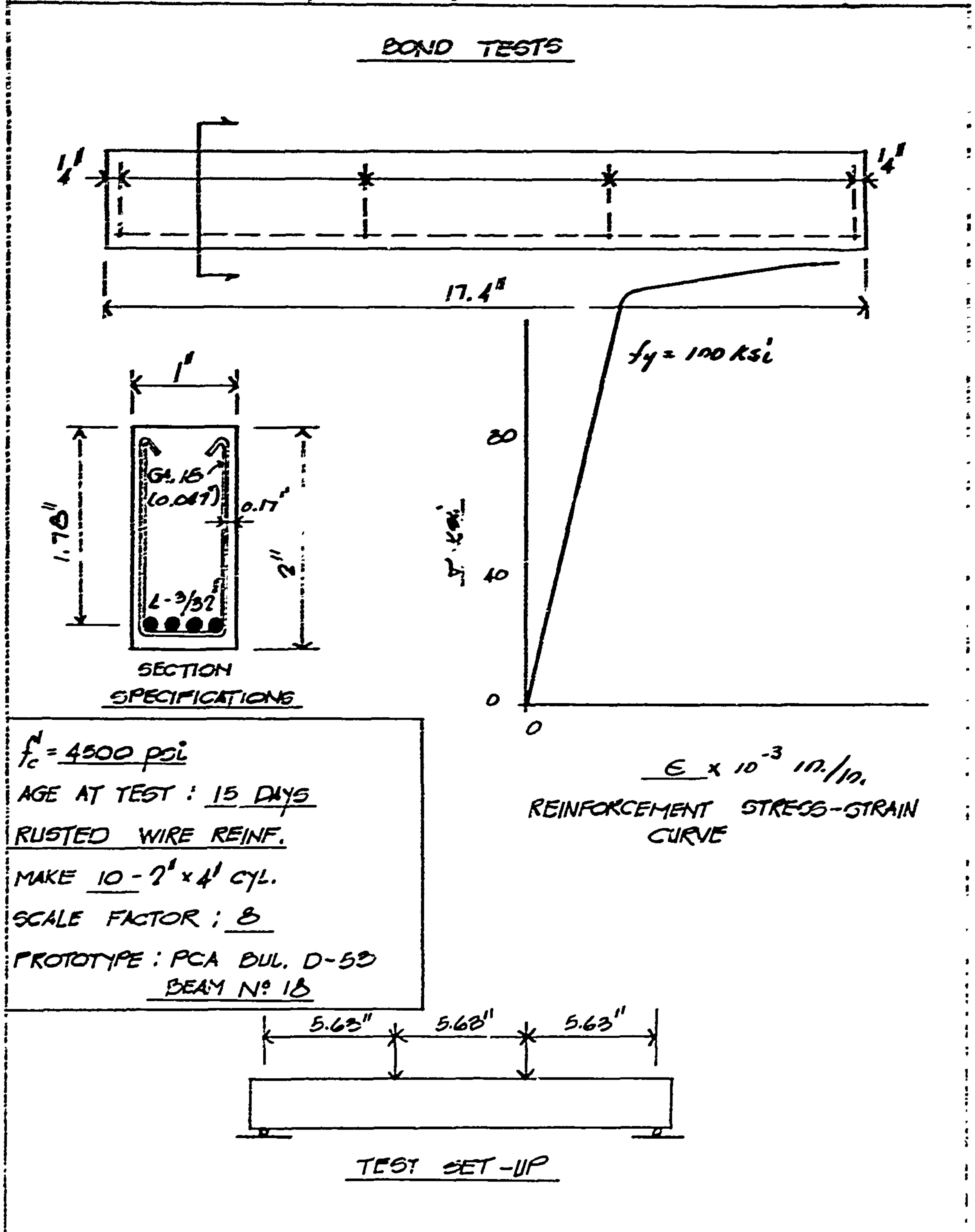


Figure 5.12

Wiss, Janney, Elstner & Associates	O.C.E. - O.C.D.	SHEET NUMBER
MADE BY: H.S. · DATE: 5/29/65 · CHECKED BY:	MICRO-CONCRETE MODEL STUDIES	JOB NUMBER: 3385

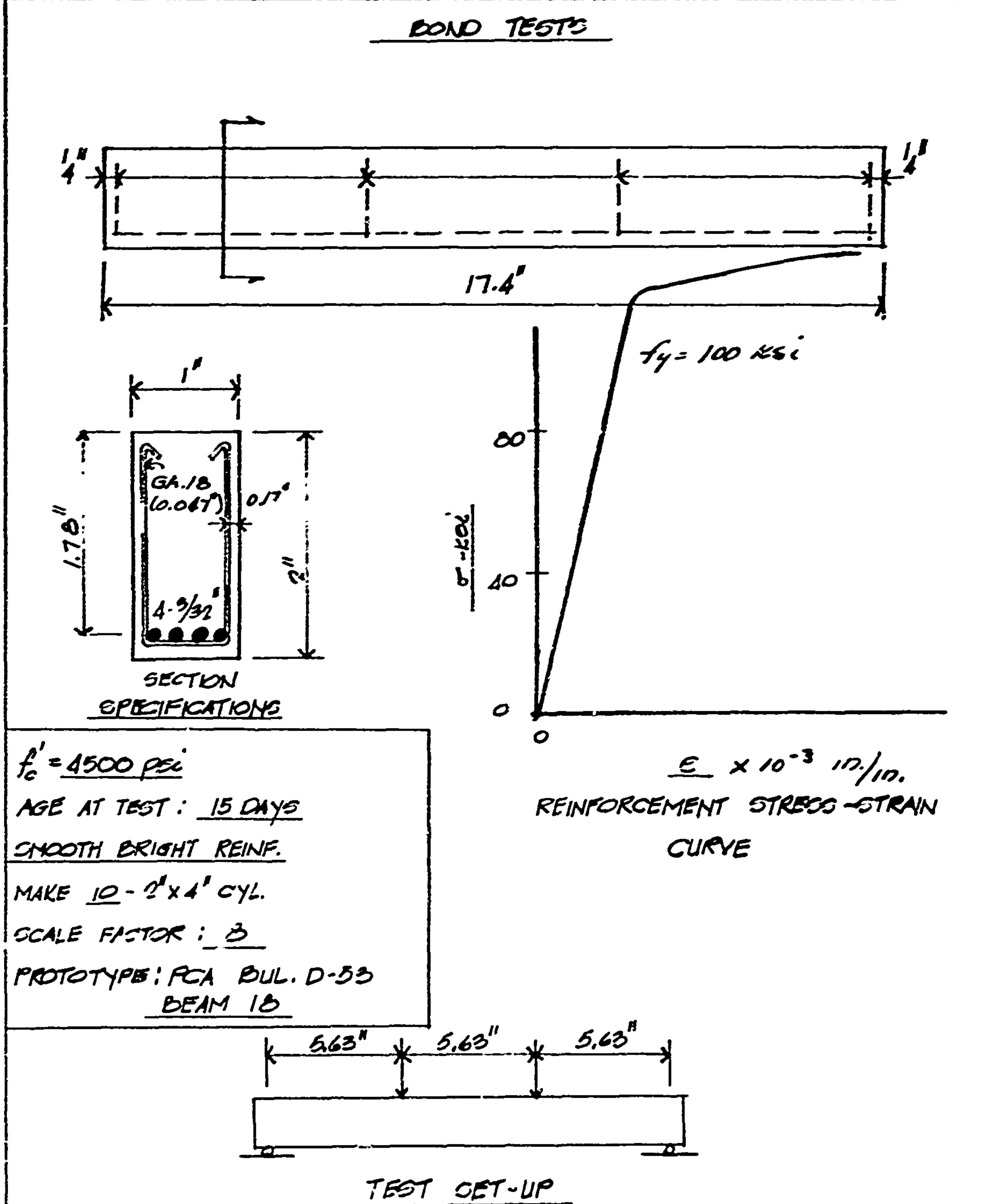
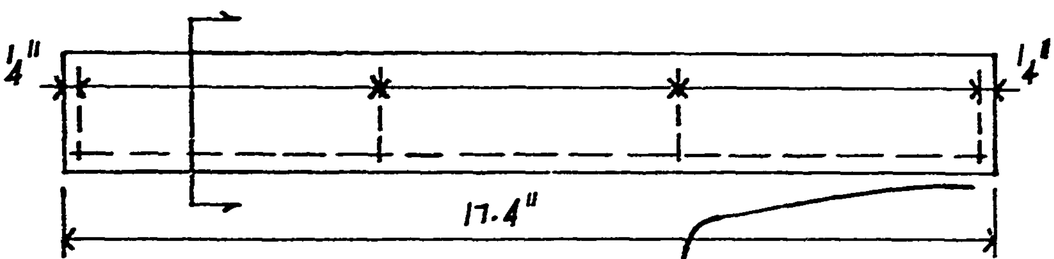


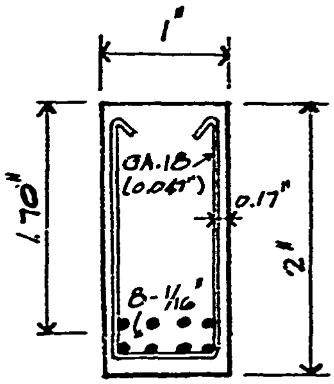
Figure 5.13

1000 H 10

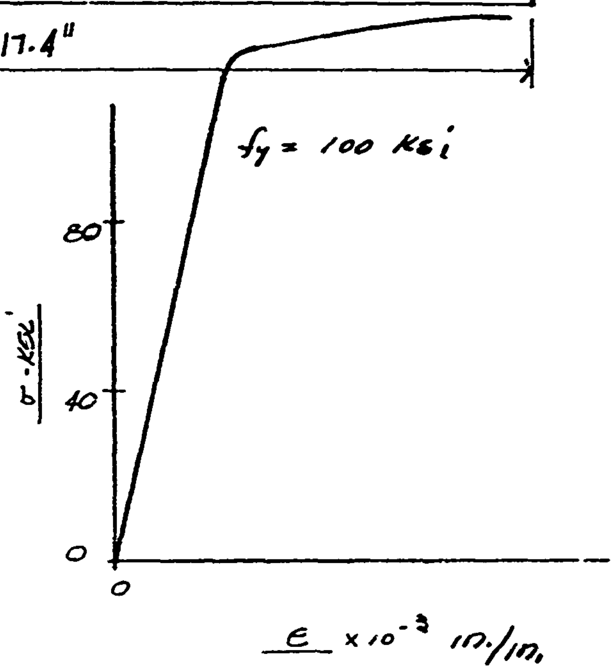
Wiss, Janney, Elstner & Associates 870 Northwest Highway • Des Plaines, Illinois	O.C.E. - O.C.D MICRO-CONCRETE MODEL STUDIES	SHEET NUMBER
MADE BY HS — DATE 5/29/65 — CHECKED BY		JOB NUMBER 3385

BOND TESTS



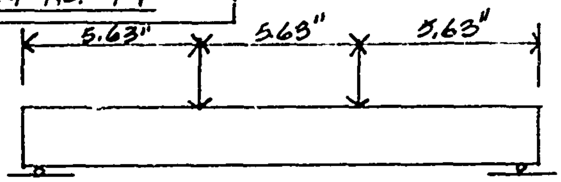


SECTION SPECIFICATIONS



REINFORCEMENT STRESS-STRAIN CURVE

f'_c = 5000 psi
AGE AT TEST: 15 DAYS
RUSTED WIRE REINF.
MAKE 10 - 2" x 4" CYL.
SCALE FACTOR: 8
PROTOTYPE: PCA BUL. D-53 BEAM NO. 19



TEST SET-UP

Figure 5.14

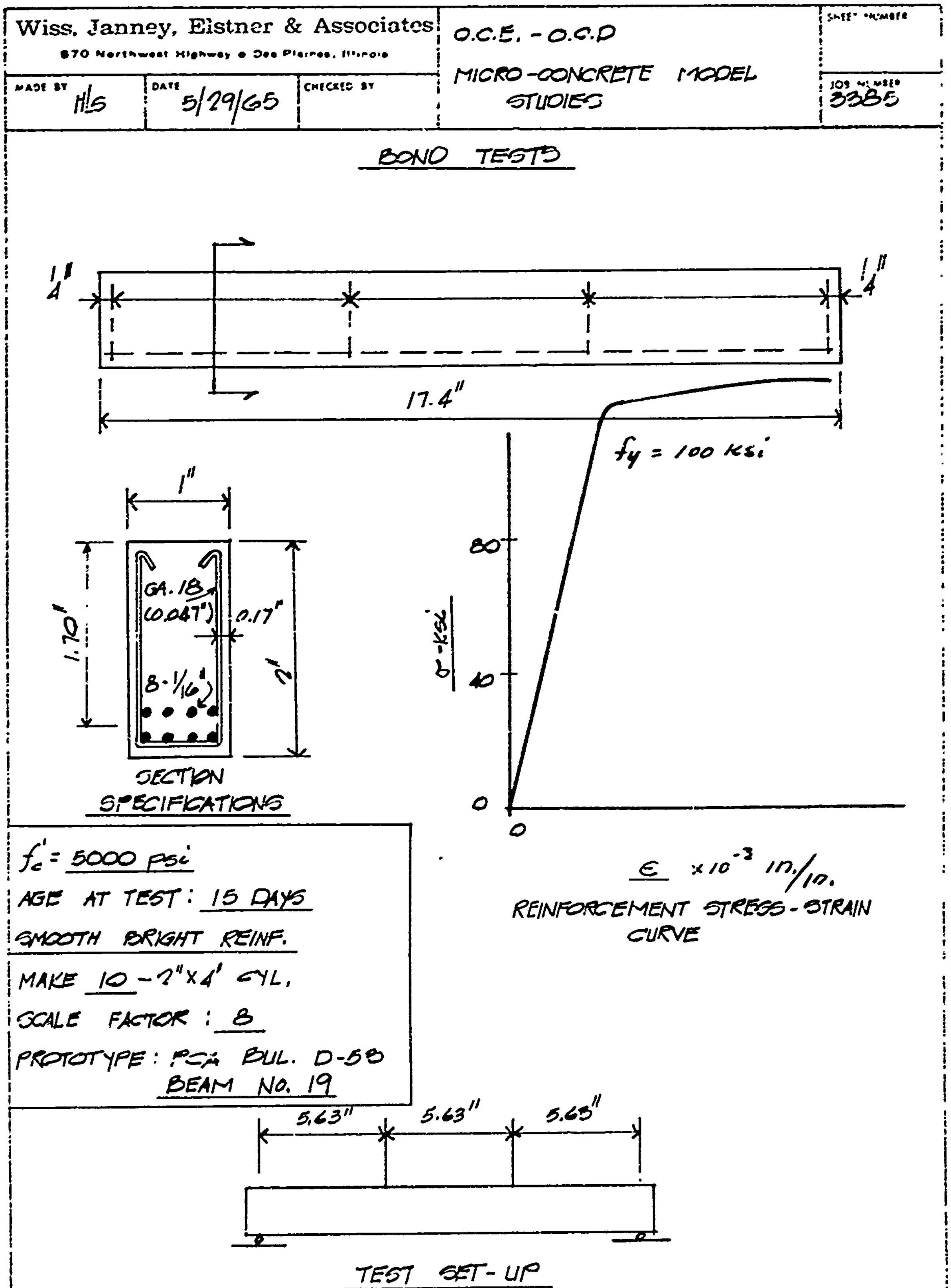
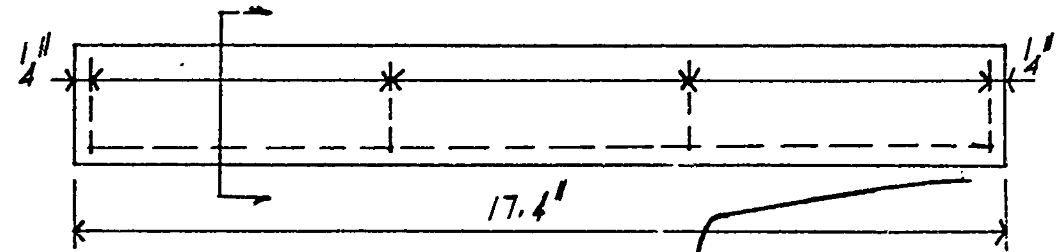
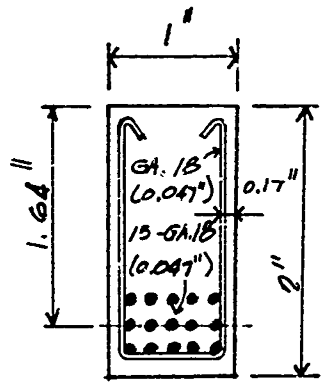

Wiss, Janney, Elstner & Associates
870 Northwest Highway • Des Plaines, Illinois
MADE BY HS
DATE 5/29/65
CHECKED BY
O.C.E. - O.C.D
MICRO-CONCRETE MODEL STUDIES
SHEET NUMBER
JOB NUMBER 3385
BOND TESTS
1/4"
1/4"
17.4"
1"
GA. 18 (0.047")
0.17"
1.70"
2"
8-1/16"
SECTION SPECIFICATIONS
f_y = 100 ksi
80
40
0
0
σ - ksi
ε x10⁻³ in./in.
REINFORCEMENT STRESS-STRAIN CURVE
f'_c = 5000 psi
AGE AT TEST: 15 DAYS
SMOOTH BRIGHT REINF.
MAKE 10 - 2"x4" CYL.
SCALE FACTOR: 8
PROTOTYPE: PCA BUL. D-58 BEAM NO. 19
5.63"
5.63"
5.63"
TEST SET-UP

Figure 5.15

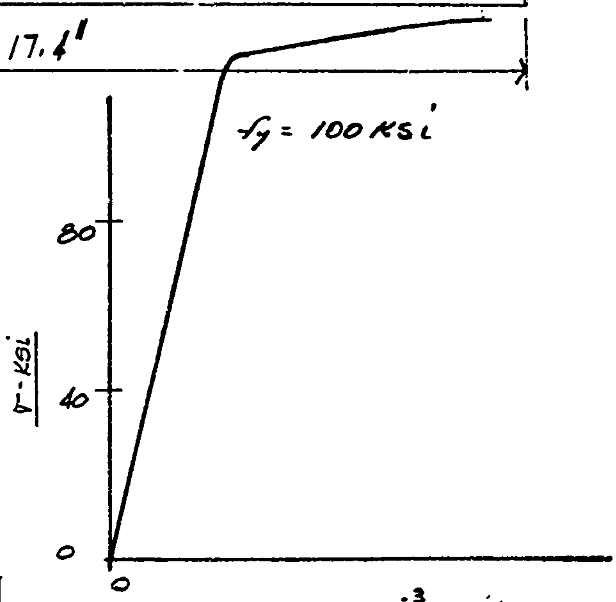
Wiss, Janney, Elstner & Associates 870 Northwest Highway • Des Plaines, Illinois	O.C.E. - O.C.D. MICRO-CONCRETE MODEL STUDIES	SHEET NUMBER
MADE BY HLS · DATE 5/29/65 · CHECKED BY		JOB NUMBER 3385

BOND TESTS



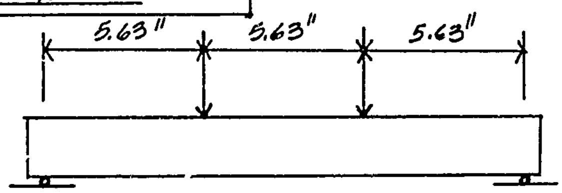


SECTION SPECIFICATIONS



REINFORCEMENT STRESS-STRAIN CURVE

f'_c = 4500 psi
AGE AT TEST : 15 DAYS
RUSTED WIRE REINF.
MAKE 10 - 2" x 4" CYL.
SCALE FACTOR : 8
PROTOTYPE : PCA BUL. D-53
BEAM No 20



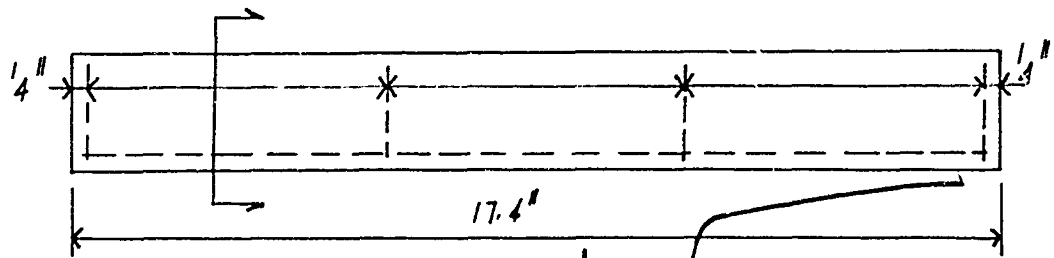
TEST SET-UP

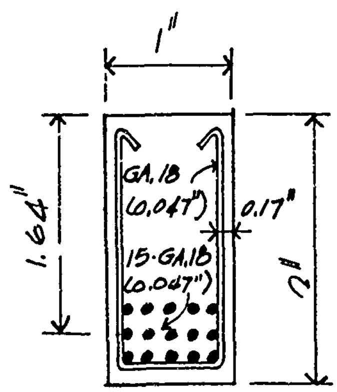
Figure 5.16

1000 H 10

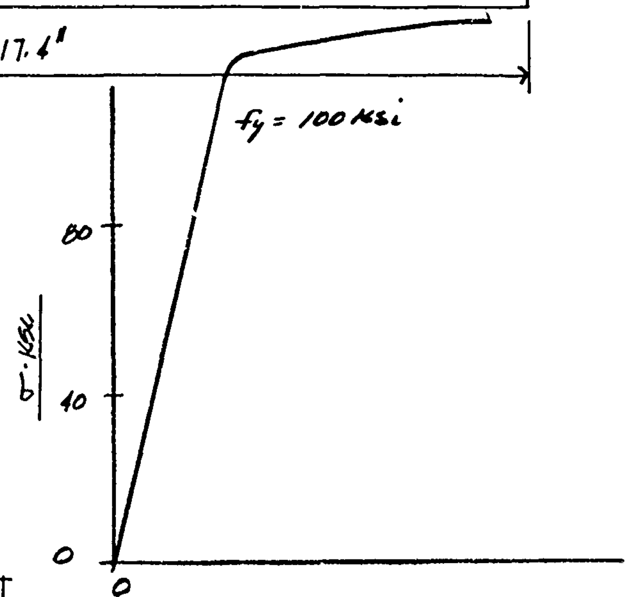
Wiss, Janney, Elstner & Associates 870 Northwest Highway • Des Plaines, Illinois	O.C.E. - O.C.D. MICRO-CONCRETE MODEL STUDIES	SHEET NUMBER
MADE BY HLS · DATE 5/29/65 · CHECKED BY		JOB NUMBER 3385

BOND TESTS





SECTION SPECIFICATIONS



REINFORCEMENT STRESS-STRAIN CURVE

f'_c = 4500 psi

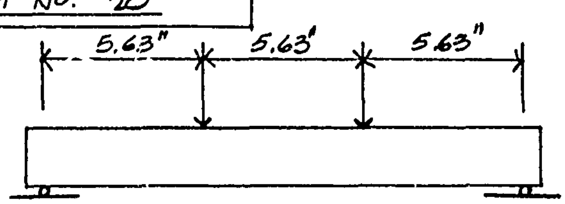
AGE AT TEST : 15 DAYS

SMOOTH BRIGHT REINF.

MAKE 10 - 2" X 4" CYL.

SCALE FACTOR : 8

PROTOTYPE : PCA BUL. D-55
BEAM NO. 20



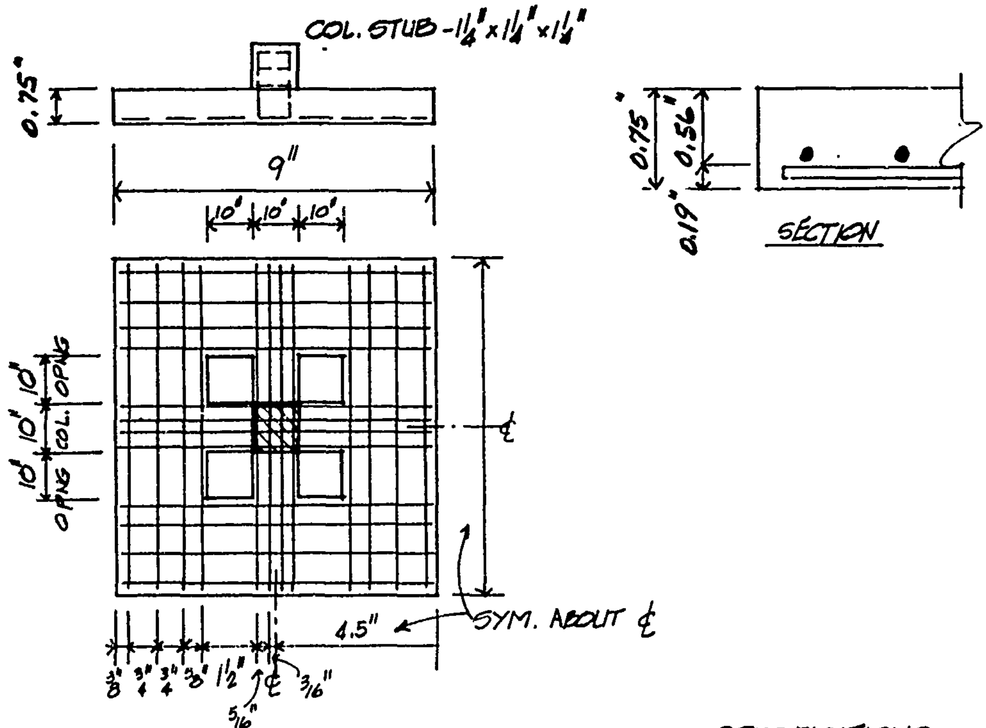
TEST SET-UP

Figure 5.17

1000 H 10

Wiss, Janney, Elstner & Associates 670 Northwest Highway • Des Plaines, Illinois	O.C.E. – O.C.D. MICRO-CONCRETE MODEL STUDIES	SHEET NUMBER
MADE BY HLG · DATE 6/1/65 · CHECKED BY		JOB NUMBER 3385

PUNCHING SHEAR TESTS



PLAN of REINF.

SPECIFICATIONS

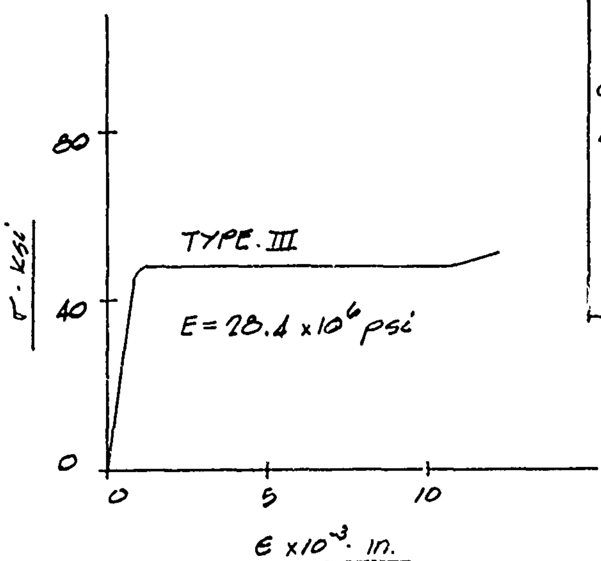
$f_c' = 3500$ psi

SCALE FACTOR: 8

COL. REINF.: 7/64" WIRE

SLAB REINF.: 12 - 5/64" WIRE EA. WAY

PROTOTYPE: PCA BUL. D-47
SLAB H-13
4 - 10" SQ. OPNGS. AS SHOWN

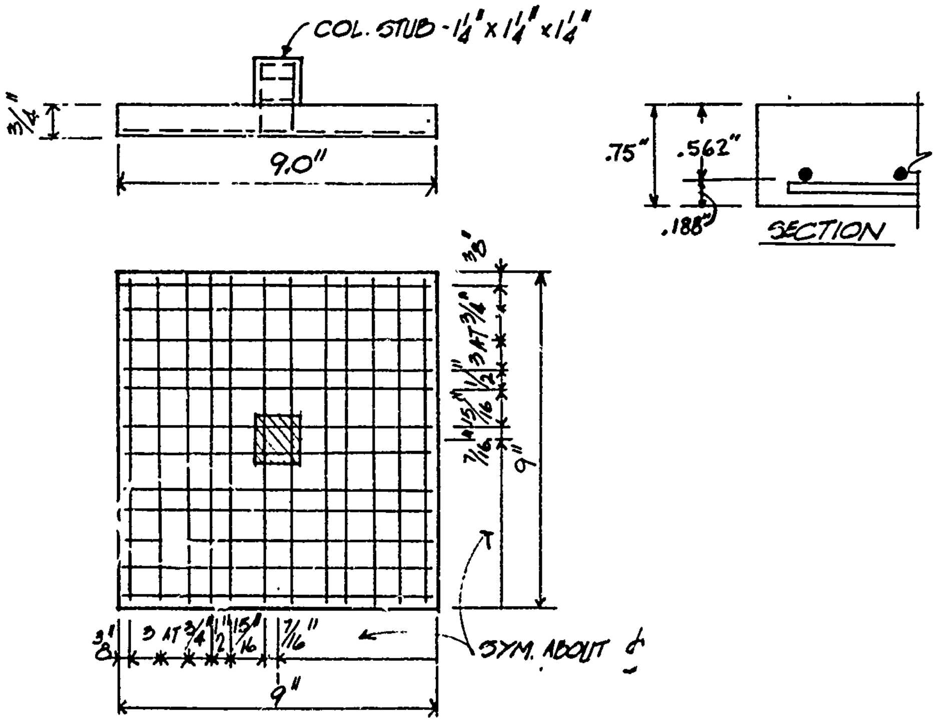


REINFORCEMENT STRESS-STRAIN CURVE

Figure 5.18

Wiss, Janney, Elstner & Associates 870 Northwest Highway • Des Plaines, Illinois	O.C.E. - O.C.D.	SHEET NUMBER
MADE BY: J.R.J. · DATE: 5-21-65 · CHECKED BY:	MICRO-CONCRETE MODEL STUDIES	JOB NUMBER: 3385

PUNCHING SHEAR TESTS



PLAN of REINF.

SPECIFICATIONS

$f'_c = 4000$ psi
SCALE FACTOR: 8
COL. REINF.: 7/64" WIRE
SLAB REINF.: 12 - 5/64" WIRE EA. WAY
PROTOTYPE: PCA BUL. D-47 SLAB H-1.

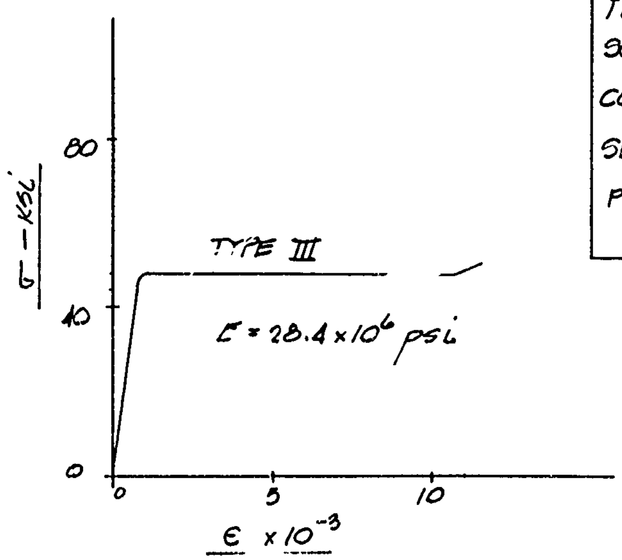
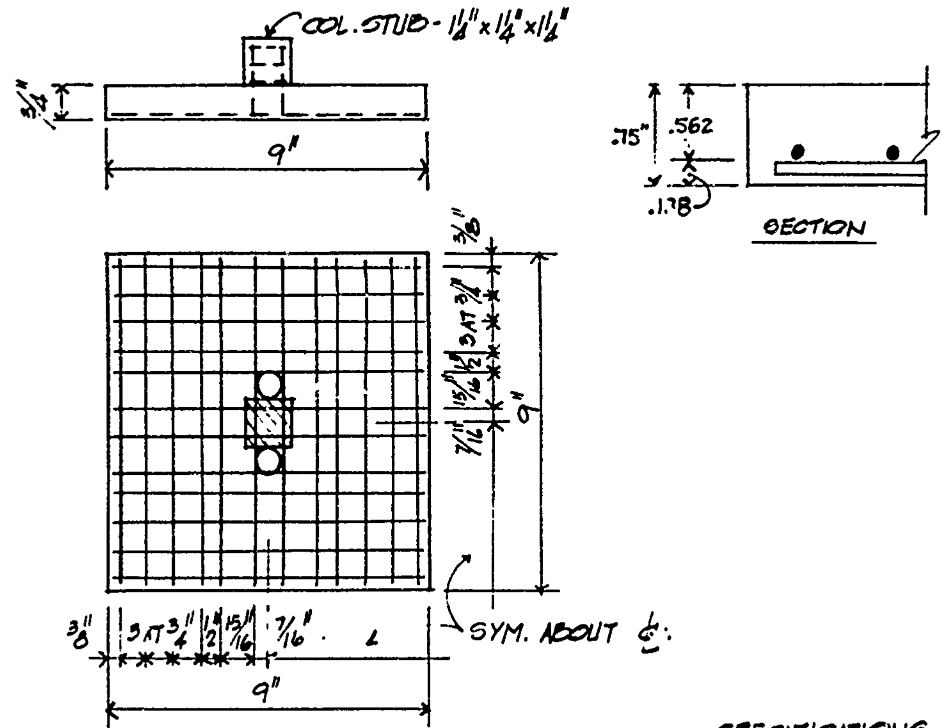


Figure 5.19

Wiss, Janney, Elstner & Associates 670 Northwest Highway • Des Plaines, Illinois	O.C.E. -O.C.D. MICRO-CONCRETE MODEL STUDIES	SHEET NUMBER
MADE BY H.S. — DATE 6/1/65 — CHECKED BY		JOB NUMBER 3385

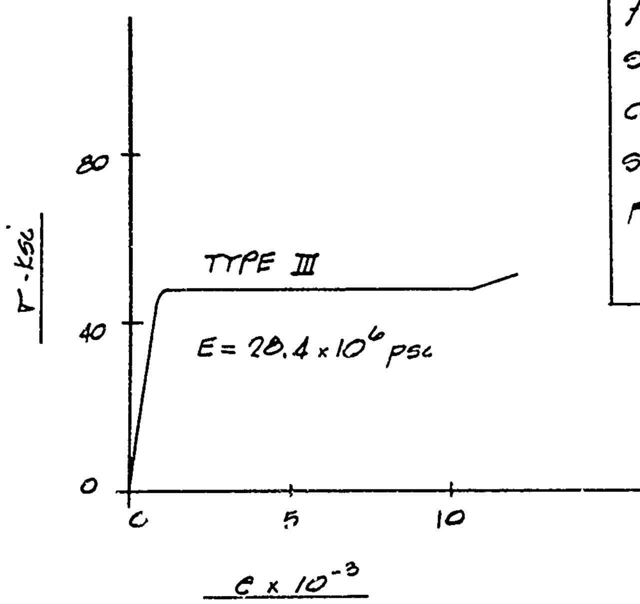
PUNCHING SHEAR TESTS



PLAN of REINF

SPECIFICATIONS

f'_c = 4000 psi
SCALE FACTOR: 8
COL. REINF. : 7/64" WIRE
SLAB REINF: 12 - 5/64" WIRE EA. WAY
PROTOTYPE : PCA BUL. D-47
SLAB H-8
2 - 5" φ OPNGS. AS SHOWN

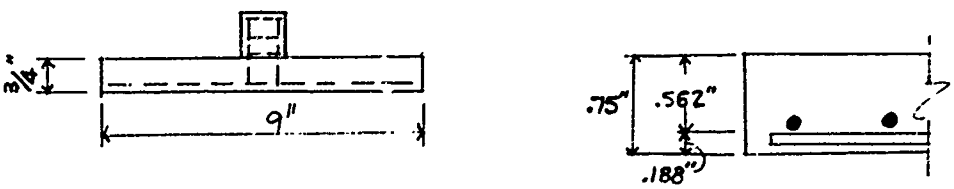


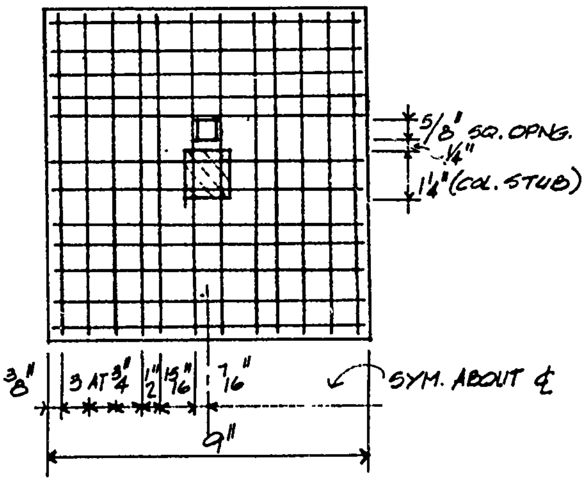
REINFORCEMENT STRESS-STRAIN CURVE

Figure 5.20

Wiss, Janney, Elstner & Associates 870 Northwest Highway • Des Plaines, Illinois	O.C.E.-O.C.D MICRO-CONCRETE MODEL STUDIES	SHEET NUMBER
MADE BY HLS · DATE 6/1/65 · CHECKED BY		JOB NUMBER 3386

PUNCHING SHEAR TESTS





PLAN of REINF.

SPECIFICATIONS

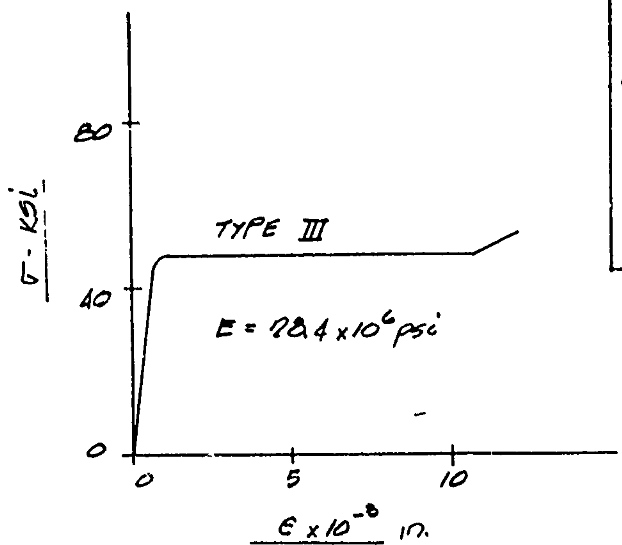
f'_c = 3500 psi

SCALE FACTOR : 8

COL. REINF. : 7/64" WIRE

SLAB REINF. : 12-5/64" WIRE-EA. WAY

PROTOTYPE : PCA BUL. D-47
SLAB H-9
1-5" SQ. OPNG. AS SHOWN



REINFORCEMENT STRESS-STRAIN CURVE

Figure 5.21

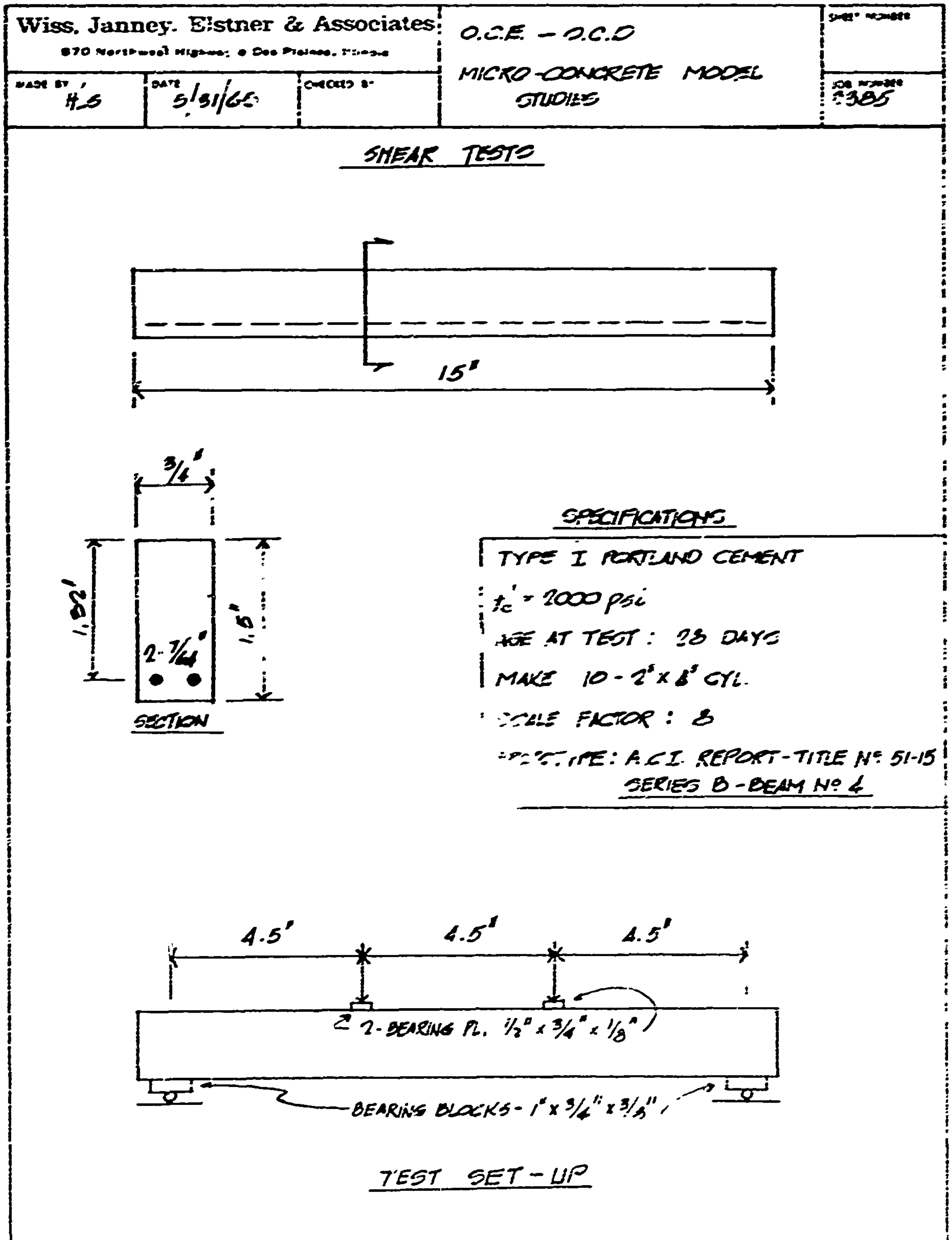
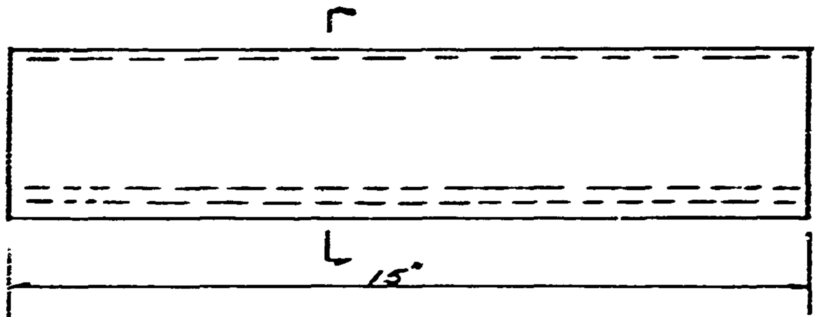
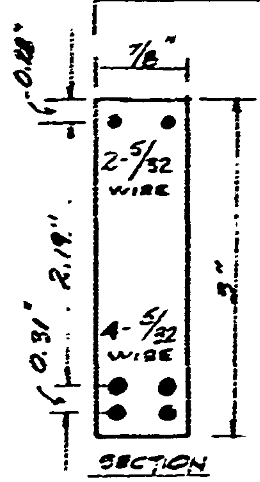

Wiss, Janney, Elstner & Associates
O.C.E. – O.C.D
MICRO-CONCRETE MODEL STUDIES
MADE BY H.S
DATE 5/31/65
CHECKED BY
JOB NUMBER 5385
SHEET NUMBER
SHEAR TESTS
15"
3/4"
1.32"
1.5"
2-7/64"
SECTION
SPECIFICATIONS
TYPE I PORTLAND CEMENT
f'c = 2000 psi
AGE AT TEST: 28 DAYS
MAKE 10 - 2" x 4" CYL.
SCALE FACTOR: 8
PROTOTYPE: A.C.I. REPORT-TITLE No 51-15 SERIES B-BEAM No 4
4.5"
4.5"
4.5"
2-BEARING PL. 1/2" x 3/4" x 1/8"
BEARING BLOCKS - 1" x 3/4" x 3/8"
TEST SET-UP

Figure 5.22

SHEAR TESTS





SPECIFICATIONS

TYPE I PORTLAND CEMENT

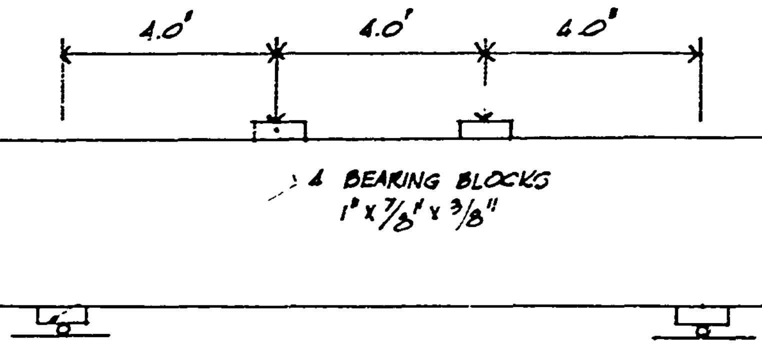
$f'_c = 3500$ psi

AGE AT TEST : 28 DAYS

MAKE 10 - 2" x 4" CYL.

SCALE FACTOR : 8

PROTOTYPE : A.C.I. REPORT - TITLE # 51-15
BEAM No. III - 27b



TEST SET-UP

Figure 5.23

Wiss, Janney, Elstner & Associates

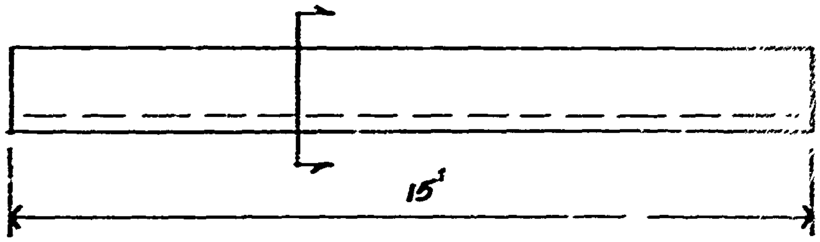
Des Plaines, Illinois

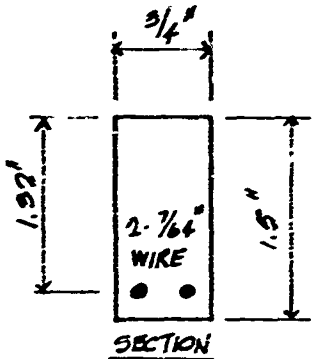
Made by H.G. | Date 5/31/65 | Checked by

O.C.E. - O.C.D.

MICRO-CONCRETE MODEL STUDIES

SHEAR TESTS





SPECIFICATIONS

TYPE I PORTLAND CEMENT

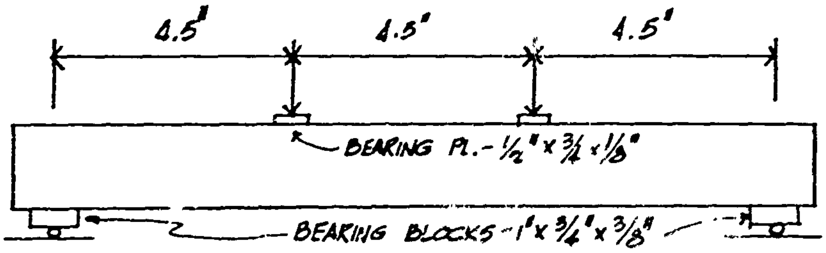
$f'_c = 3500$ psi

AGE AT TEST: 28 DAYS

MAKE 10 - 2" x 4" CYL.

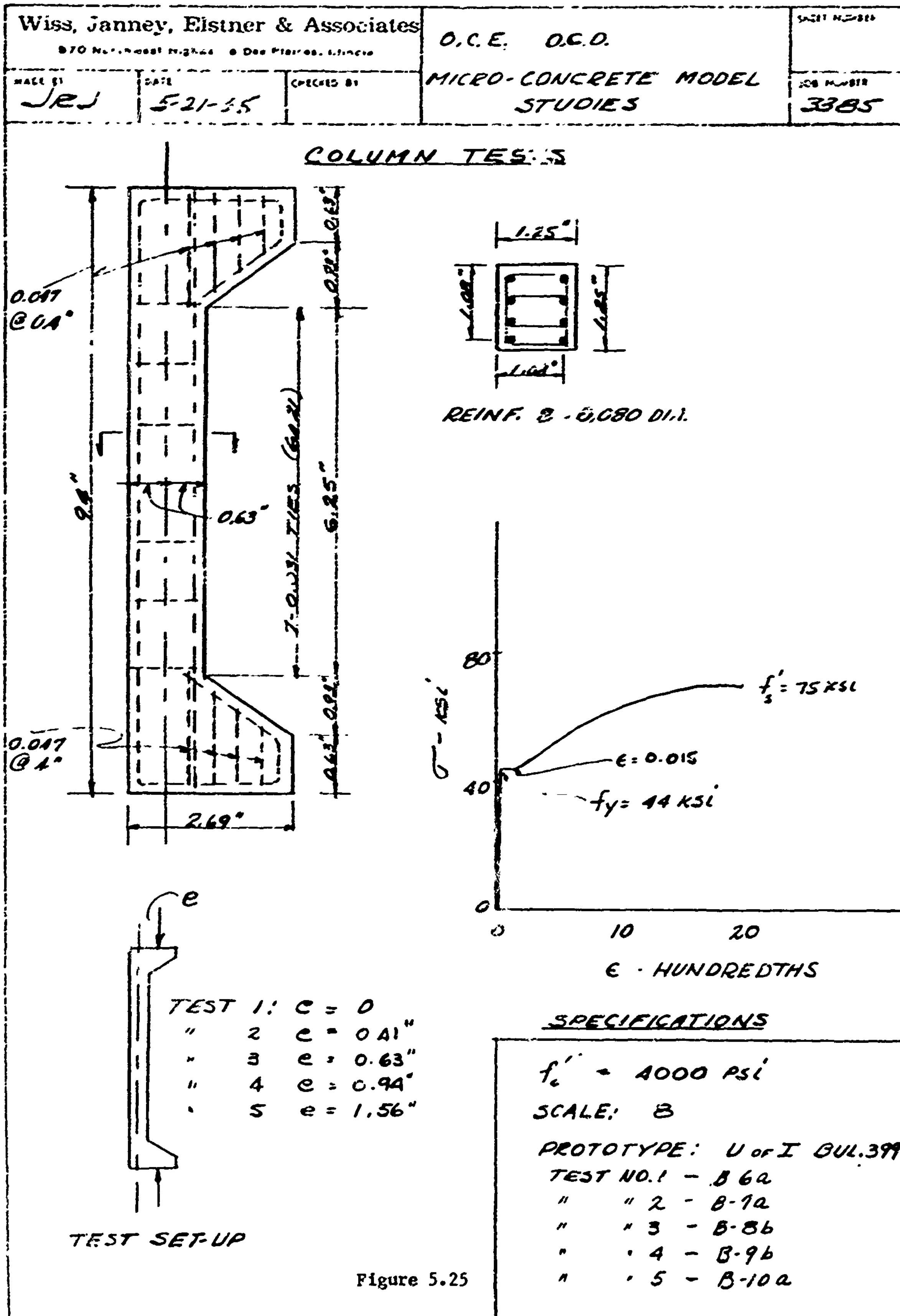
SCALE FACTOR: 8

PROTOTYPE: ACI REPORT - TITLE No. 51-15 SERIES B - BEAM No. 1.



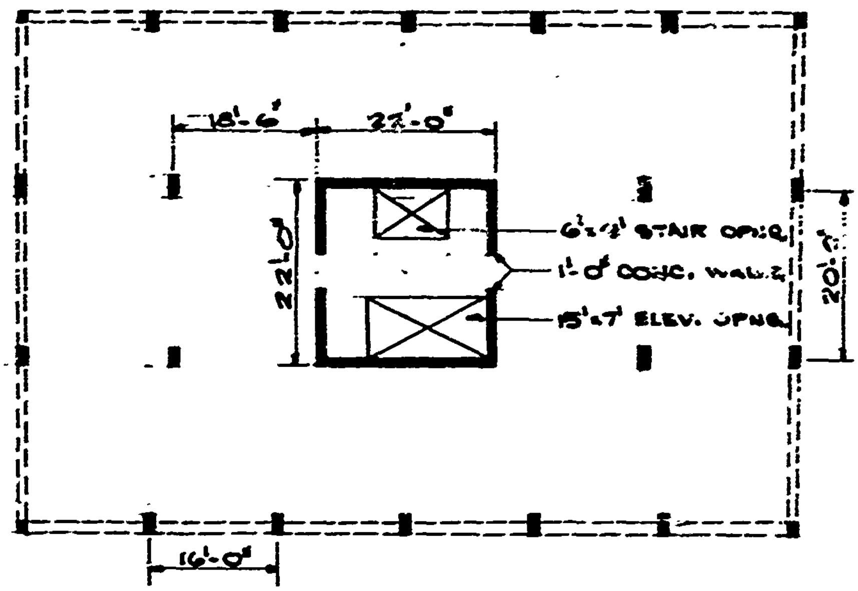
TEST SET-UP

Figure 5.24

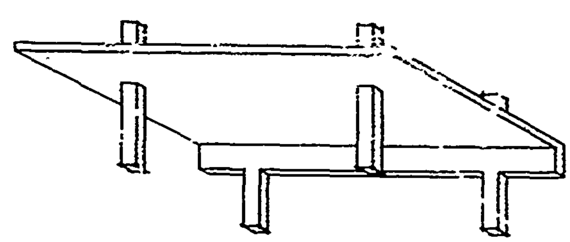


Wiss, Janney, Elstner & Associates
O.C.E. O.C.D.
MICRO-CONCRETE MODEL STUDIES
JRJ
5-21-65
3385
COLUMN TESTS
0.047 @0.4"
0.047 @.4"
0.63"
0.82"
0.63"
7-0.031 TIES (GA.21)
6.25"
2.69"
1.25"
1.08"
1.25"
1.08"
REINF. 8 - 0.080 DIA.
80
40
0
0
10
20
ε - HUNDREDTHS
σ - KSI
f'_s = 75 KSI
ε = 0.015
f_y = 44 KSI
e
TEST 1: e = 0
" 2 e = 0.41"
" 3 e = 0.63"
" 4 e = 0.94"
" 5 e = 1.56"
TEST SET-UP
SPECIFICATIONS
f'_c = 4000 PSI
SCALE: 8
PROTOTYPE: U of I BUL.399
TEST NO.1 - B 6a
" " 2 - B-7a
" " 3 - B-8b
" " 4 - B-9b
" " 5 - B-10a

Figure 5.25

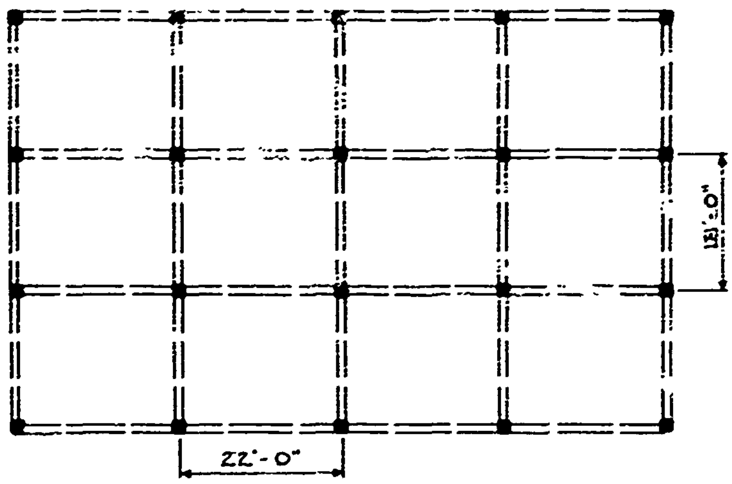


FLAT PLATE

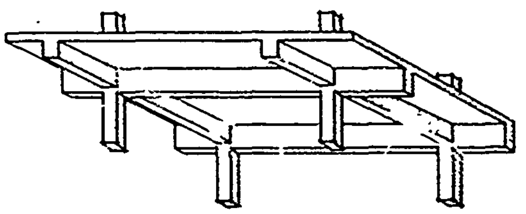


TYPE OF BUILDING - APARTMENT
COLUMN SIZE - 12" x 20"
BEAM SIZE - 12" x 18"
SLAB THICKNESS - 7"
FLOOR TO FLOOR HEIGHT - 8'-9"

Figure 5.26

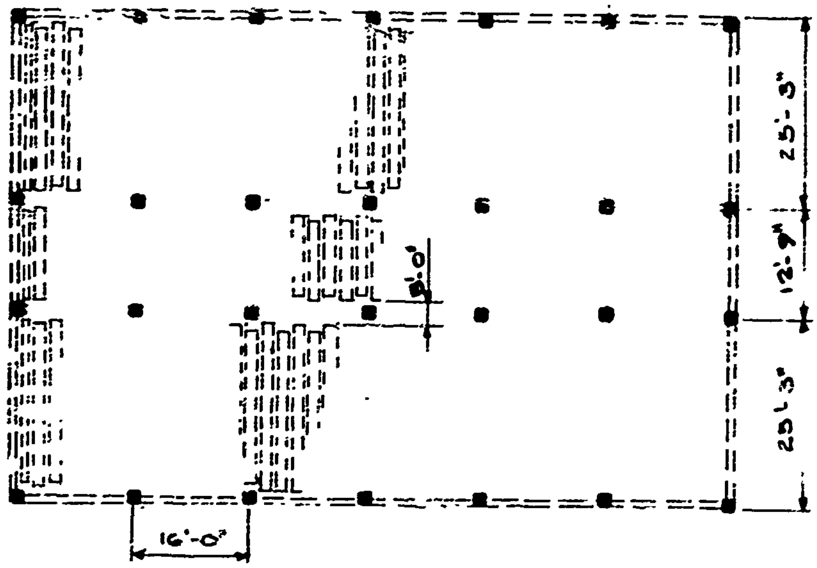


TWO WAY
SOLID SLAB

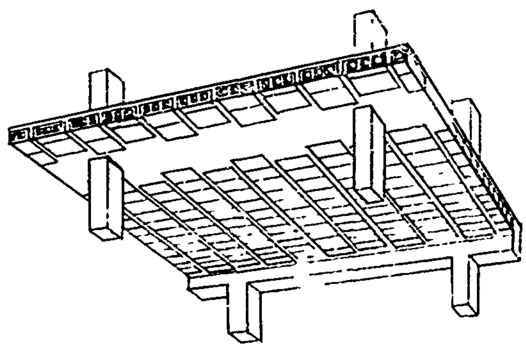


TYPE OF BUILDING - OFFICE
COLUMN SIZE - 14" x 14"
BEAM SIZE - 14" x 20"
SLAB THICKNESS - 6"
FLOOR TO FLOOR HEIGHT - 12'-0"

Figure 5.27

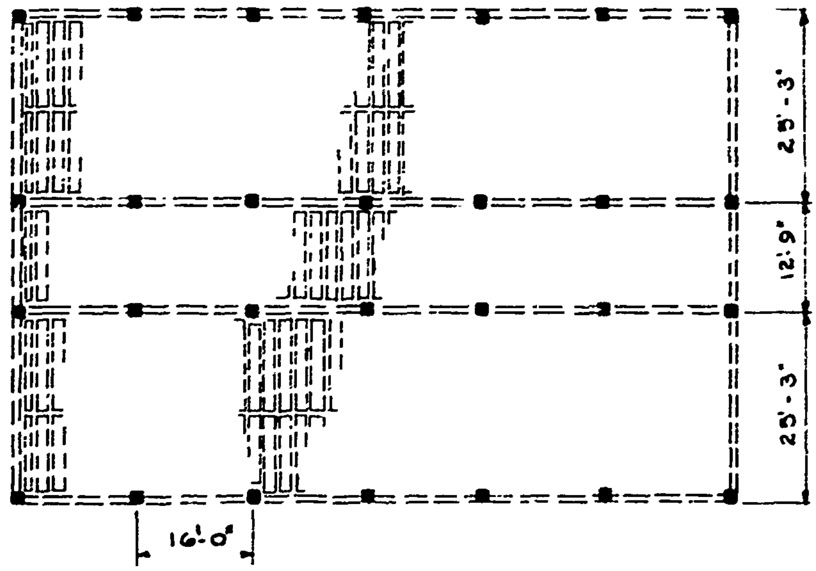


ONE WAY
SOFFIT BLOCK

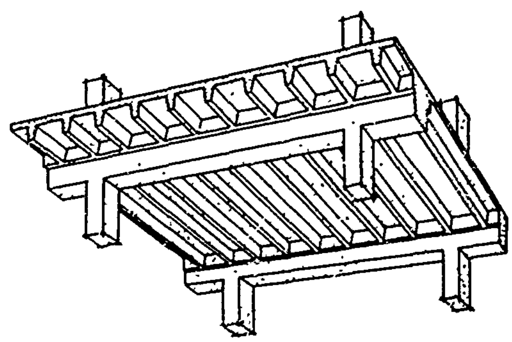


TYPE OF BUILDING - SCHOOL
COLUMN SIZE -12"x12"
BEAM SIZE -12"x20"
JOIST SIZE 5"x10"+2½"
BLOCK SIZE 16"x10"
SLAB THICKNESS 2½"
FLOOR TO FLOOR HEIGHT - 11'-0"

Figure 5.28

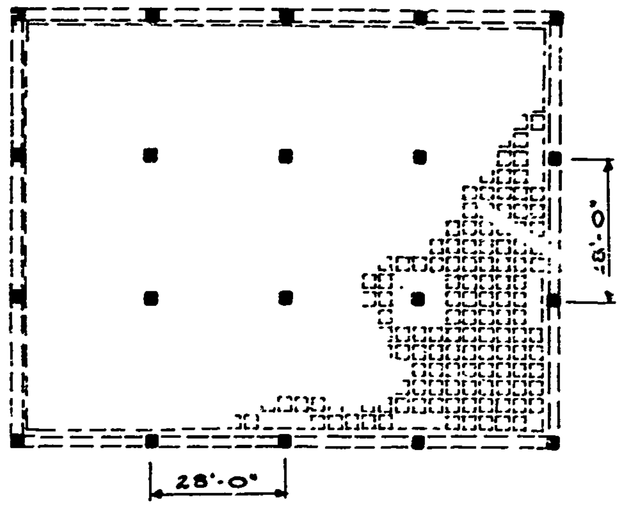


ONE WAY
PAN JOIST

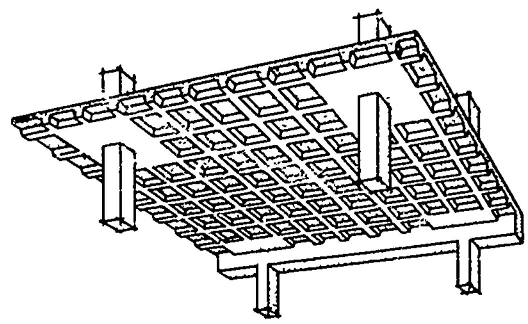


TYPE OF BUILDING - SCHOOL, APARTMENT
COLUMN SIZE - 12" x 12"
BEAM SIZE - 12" x 20"
JOIST SIZE - 5" x 10" + 2½"
PAN SIZE - 20" x 10"
SLAB THICKNESS - 2½"
FLOOR TO FLOOR HEIGHT - 11'-0"

Figure 5.29

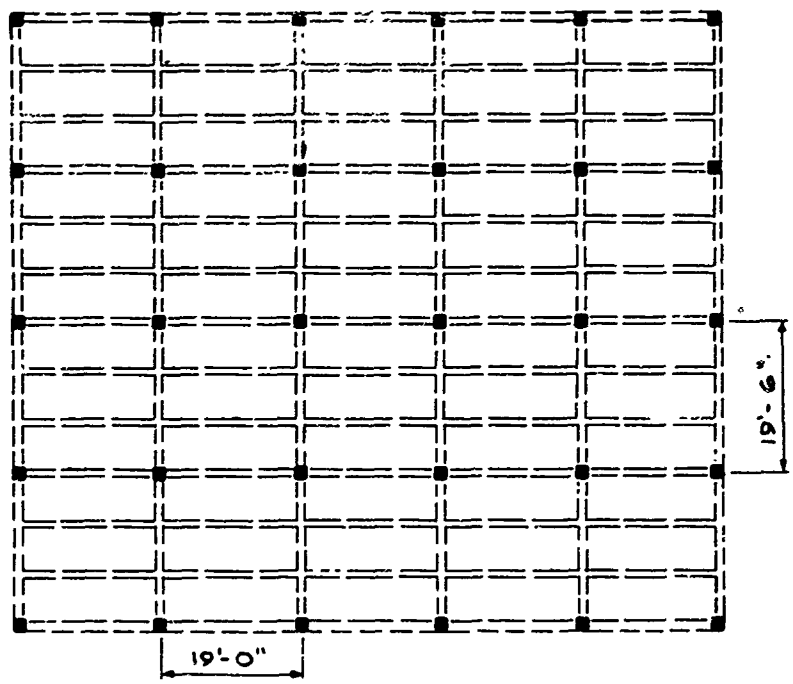


TWO WAY
WAFFLE SLAB

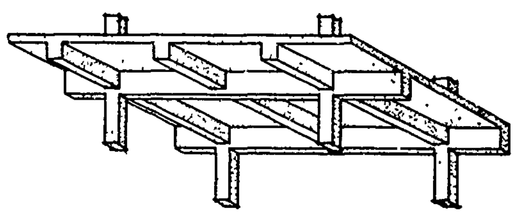


TYPE OF BUILDING - FACTORY
COLUMN SIZE 16" x 16"
RIB SIZE 6" x 12" + 4 1/2"
SLAB THICKNESS 4 1/2"
FLOOR TO FLOOR HEIGHT 12'-0"
BEAM SIZE 16" x 24"

Figure 5.30

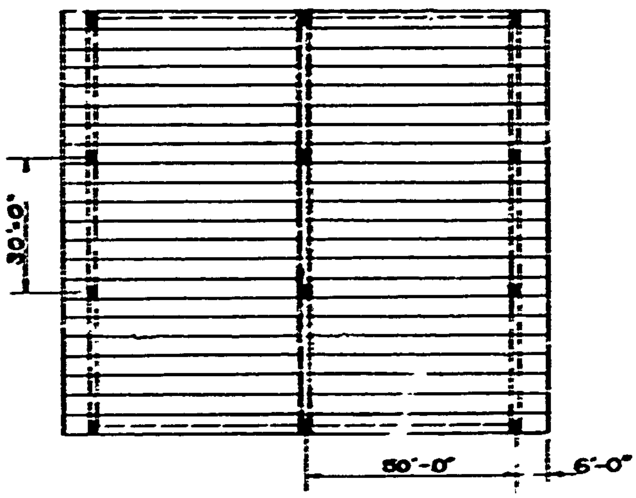


ONE WAY SLAB
BEAM & GIRDER

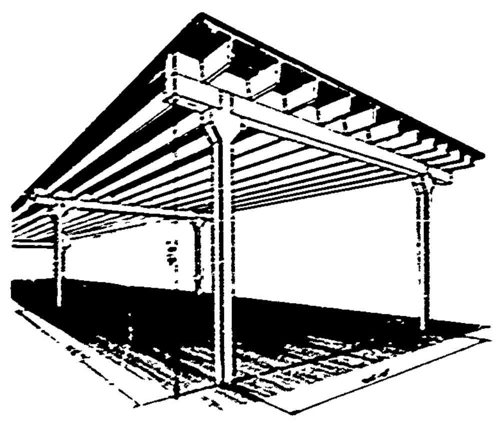


TYPE OF BUILDING - FACTORY
COLUMN SIZE 14" x 14"
BEAM SIZE 10" x 20"
GIRDER SIZE 14" x 28"
SLAB THICKNESS 4"
FLOOR TO FLOOR HEIGHT 13'-0"

Figure 5.31

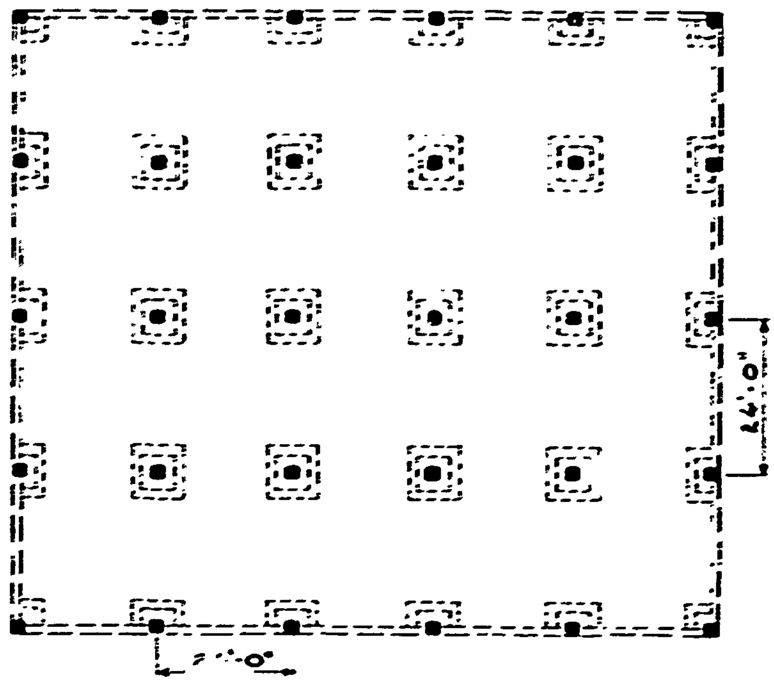


PRECAST PRE-
STRESSED DOUBLE
TEES & COMPOSITE
BEAM

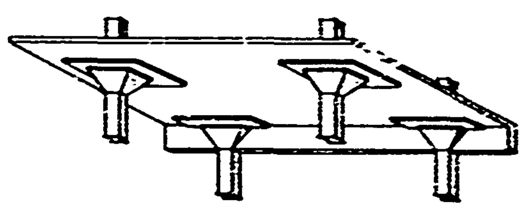


TYPE OF BUILDING - WAREHOUSE
COLUMN SIZE 12" x 12"
DOUBLE TEE 14" x 4'-0"
COMPOSITE BEAMS 12" x 20" + 14"
DECK THICKNESS 2"
COLUMN HEIGHT 18'-0"

Figure 5.32

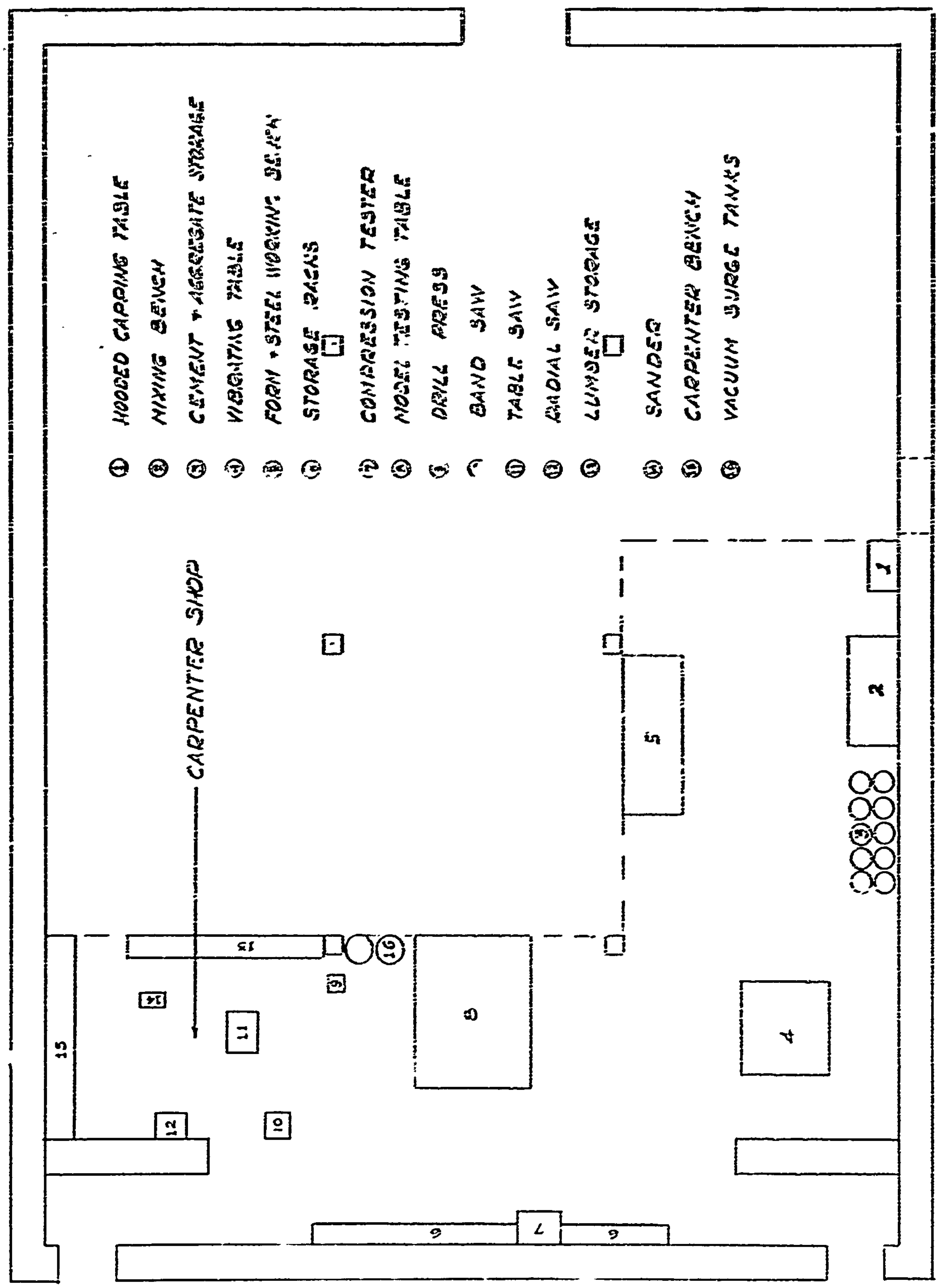


FLAT SLAB



TYPE OF BUILDING - FACTORY
COLUMN SIZE - 16" x 16"
SHEAR HEAD SIZE - 3'-0" x 3'-0"
DROP PANEL SIZE - 8'-0" x 8'-0" x 4"
SLAB THICKNESS - 8 1/2"
FLOOR TO FLOOR HEIGHT - 14'-0"
BEAM SIZE - 14" x 24"

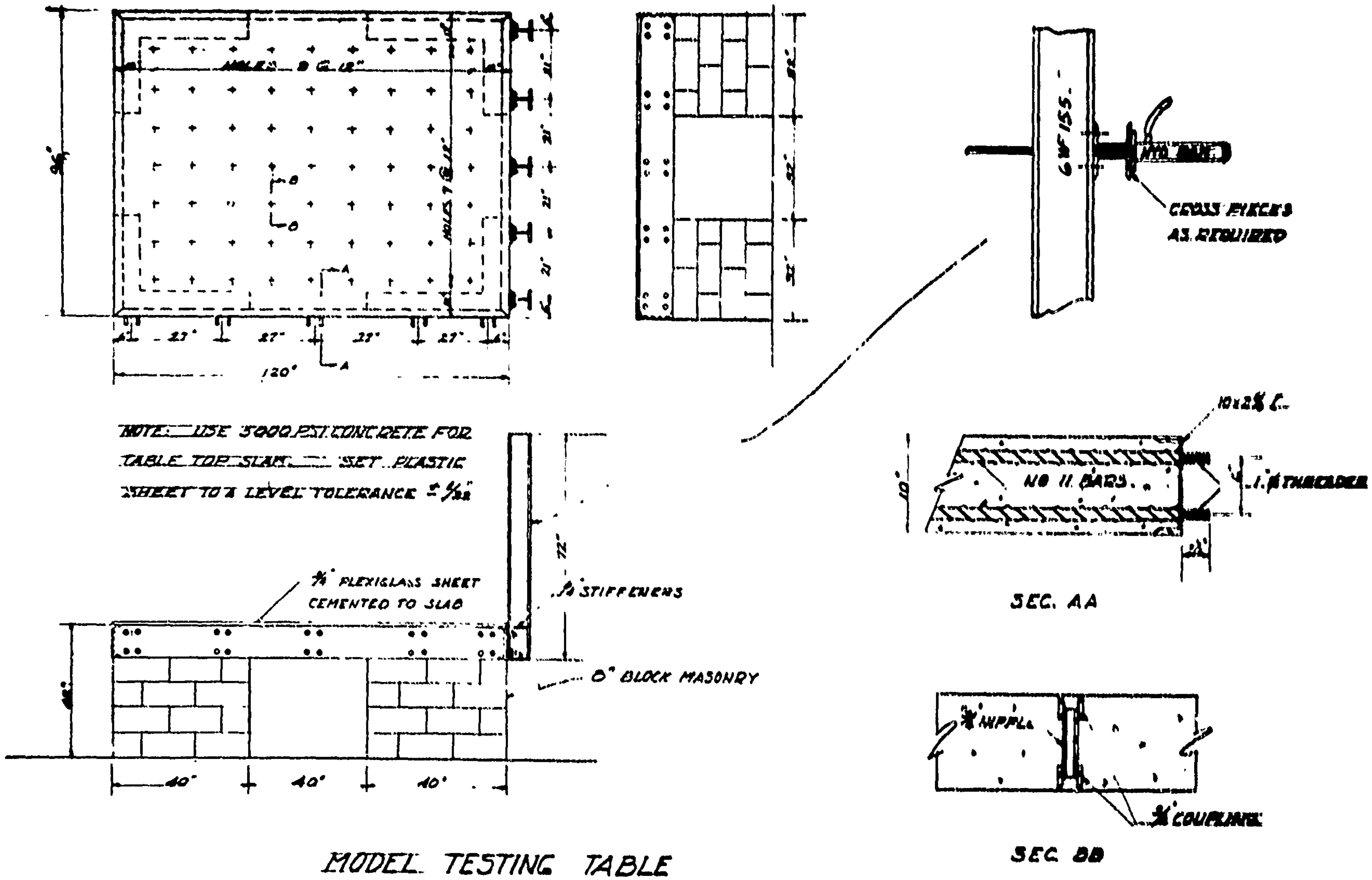
Figure 5.33



MODEL LABORATORY PLAN

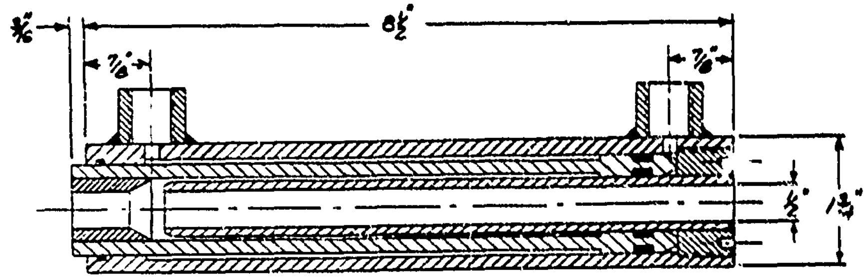
SCALE 3/32" = 1'0"

Figure 5.34

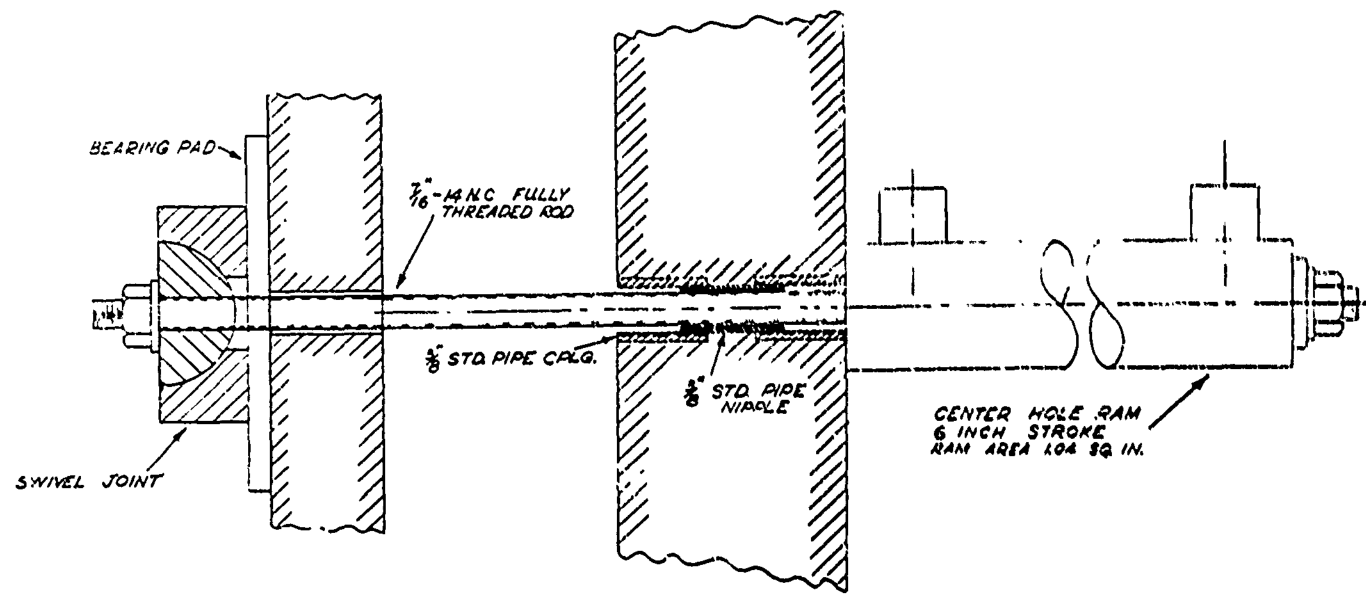


MODEL TESTING TABLE

Figure 5.35



DETAIL OF CENTER HOLE RAM



DETAILS OF
RAM AND MODEL LOADING APPARATUS

Figure 5.36

NOTE:
1. FILTER IS INSIDE RESERVOIR ON PUMP INTAKE LINE.
2. HYDRAULIC PUMP IS DIRECTLY CONNECTED TO ELECTRIC MOTOR.
3. RESERVOIR IS MOUNTED OVER PUMP FOR MORE EFFICIENT OPERATION.

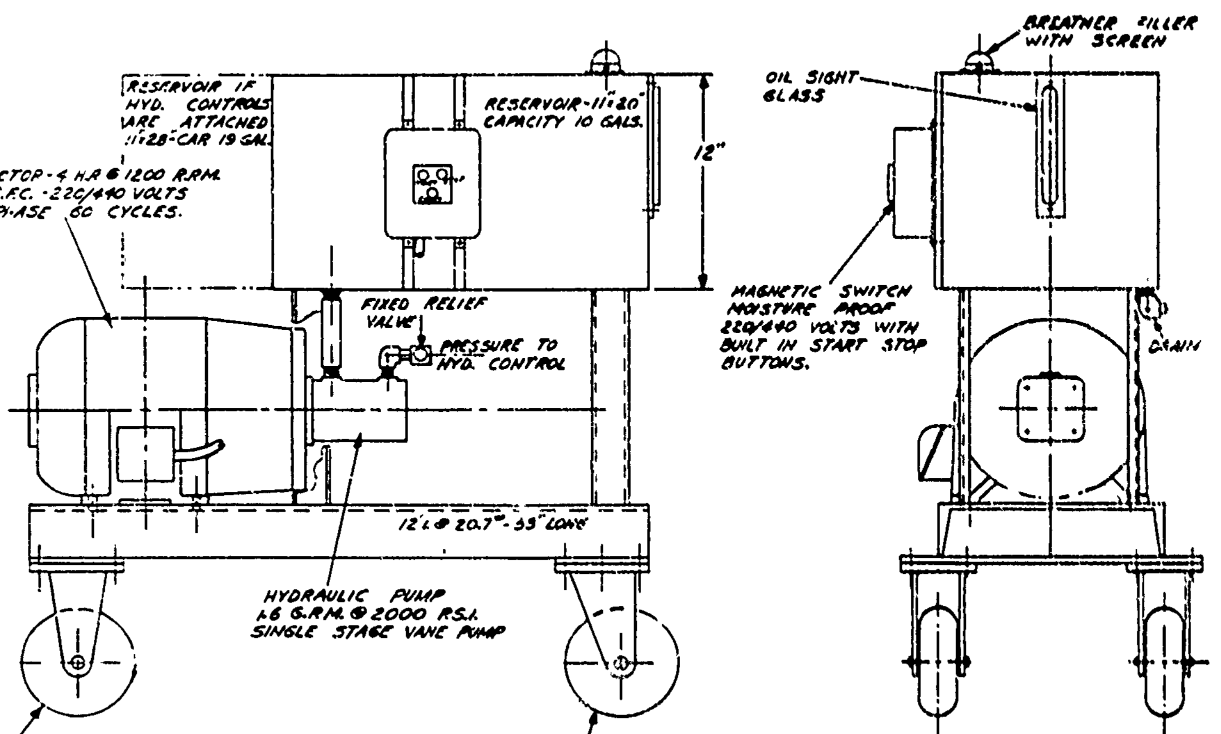


Figure 5.37

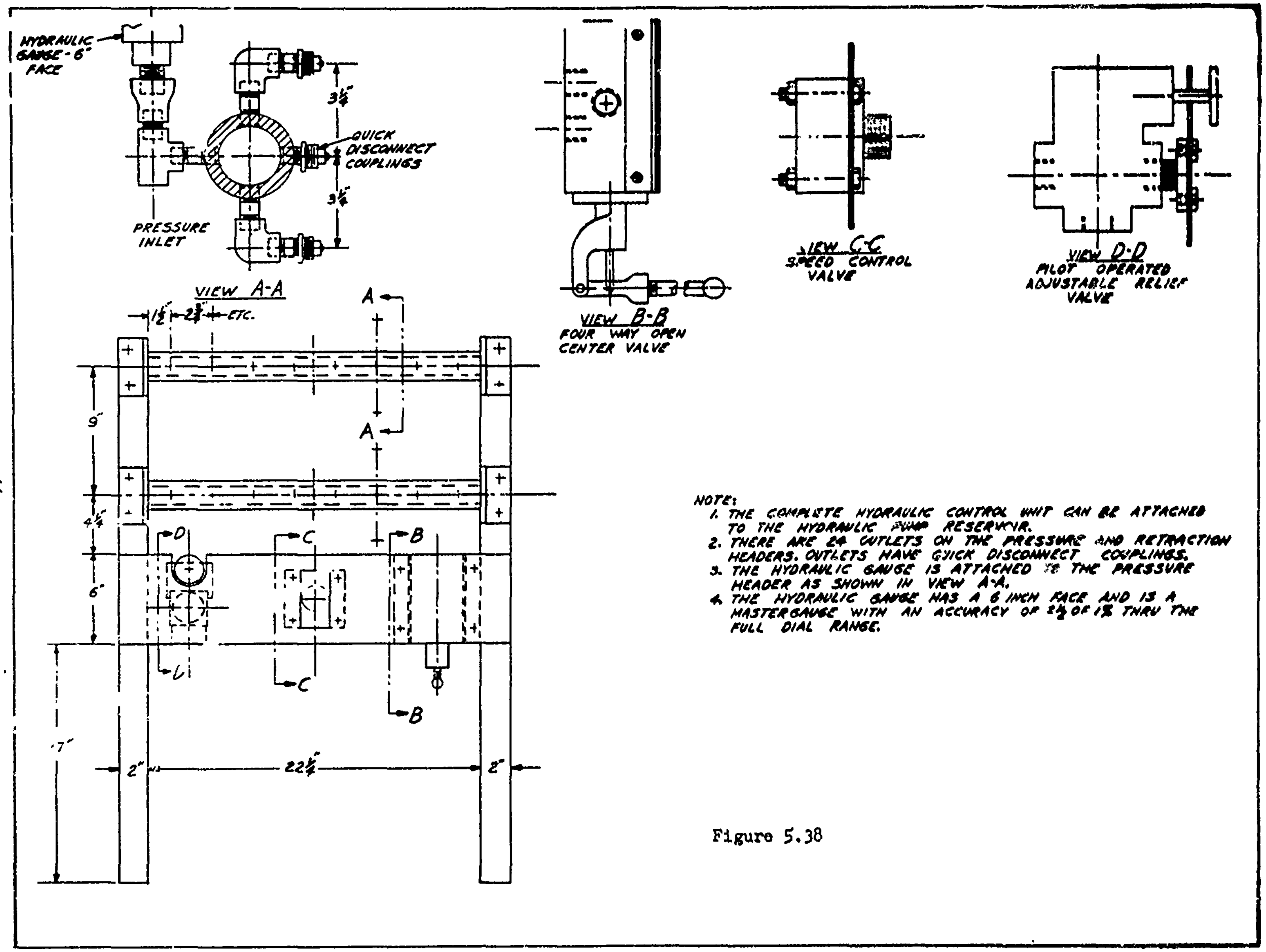
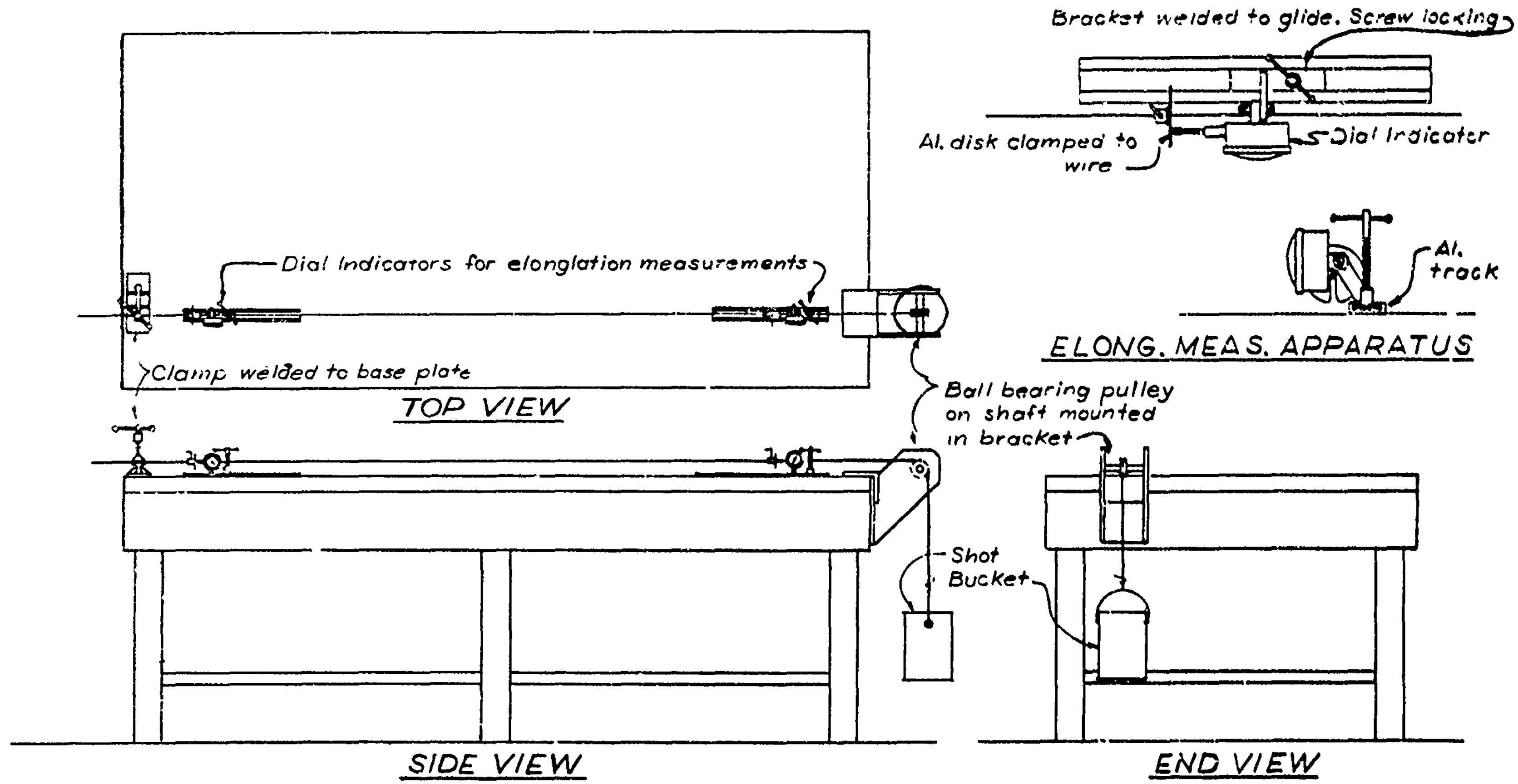

HYDRAULIC GAUGE - 6" FACE
3¼"
QUICK DISCONNECT COUPLINGS
3¼"
PRESSURE INLET
VIEW A-A
1½" 2¾" ETC.
A
A
B
B
C
C
D
D
9"
4¼"
6"
7"
2"
22½"
2"
VIEW B-B
FOUR WAY OPEN CENTER VALVE
VIEW C-C
SPEED CONTROL VALVE
VIEW D-D
PILOT OPERATED ADJUSTABLE RELIEF VALVE
NOTE:
1. THE COMPLETE HYDRAULIC CONTROL UNIT CAN BE ATTACHED TO THE HYDRAULIC PUMP RESERVOIR.
2. THERE ARE 24 OUTLETS ON THE PRESSURE AND RETRACTION HEADERS. OUTLETS HAVE QUICK DISCONNECT COUPLINGS.
3. THE HYDRAULIC GAUGE IS ATTACHED TO THE PRESSURE HEADER AS SHOWN IN VIEW A-A.
4. THE HYDRAULIC GAUGE HAS A 6 INCH FACE AND IS A MASTERGAUGE WITH AN ACCURACY OF ¼ OF 1% THRU THE FULL DIAL RANGE.

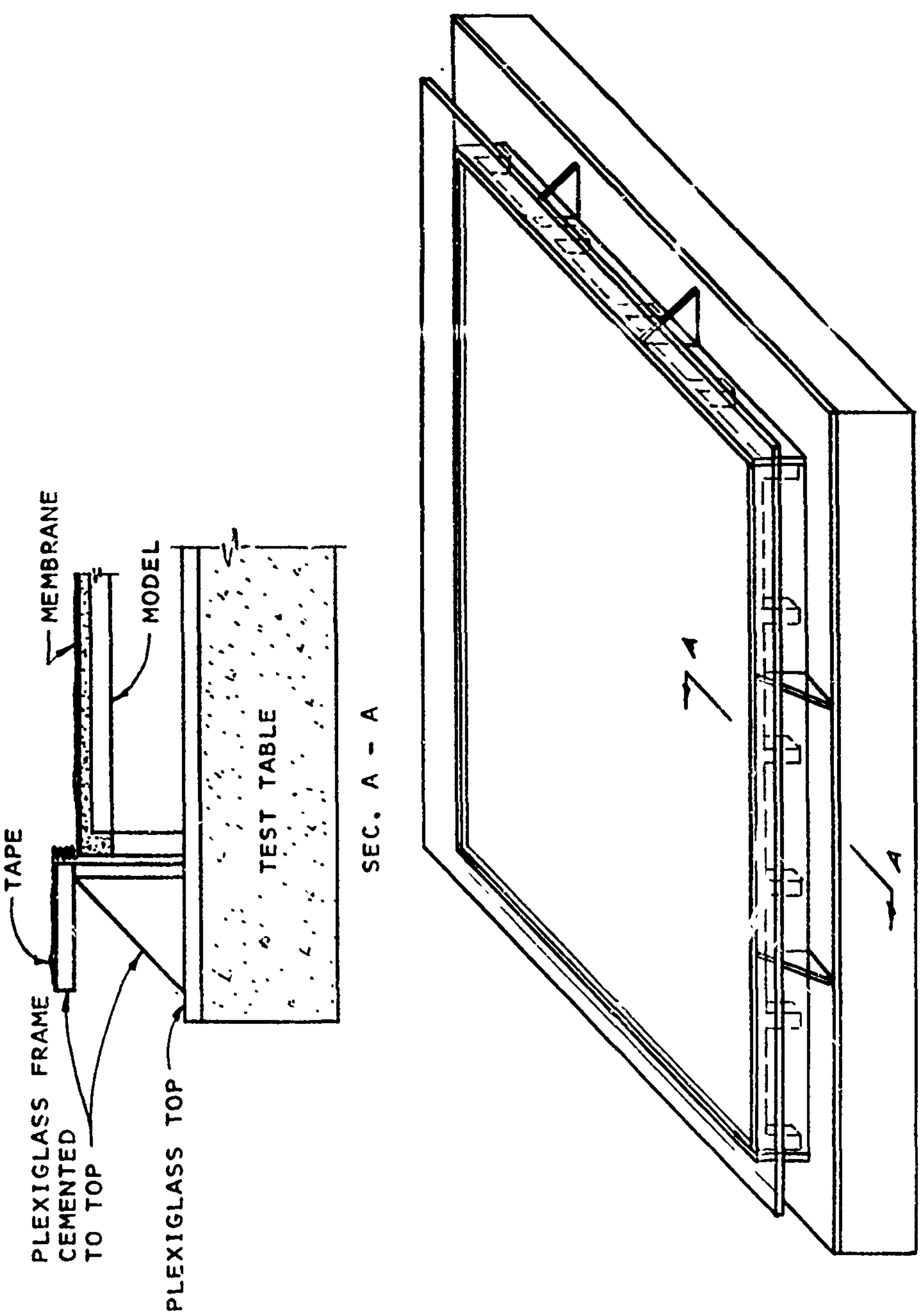
Figure 5.38



WIRE STRETCHER

Figure 5.39

Wiss, Janney, Elstner & Associates 870 Northwest Highway, Des Plaines, Illinois	O.C.E. - O.C.D. MICRO - CONCRETE MODEL STUDIES	SHEET NUMBER
MADE BY: JRJ — DATE: 6/1/65 — CHECKED BY:		JOB NUMBER: 3385



SCHEMATIC VACUUM LOAD FRAME

Figure 5.40

SELECTED BIBLIOGRAPHY

GENERAL

LITERATURE REVIEW

PUBLISHED ARTICLES

1-A. C.U.R. (Committee of Concrete Research), "The Design of Slabs According to the Yield-Line Theory", Rapport 26A, December 1962 (In Dutch with English summary)

Results and Conclusions

In order to insure serviceability of structures designed by yield-line theory, several rules have been proposed.

1. Deflections will be limited by expressing the slab thickness as a function of the span, steel grade and edge conditions.

2. Because the yield-line theory requires that hinges be formed, conditions are attached to both the minimum and maximum percentage of reinforcement.

3. It is proposed that the ratio between negative and positive bending moment in continuous or fixed slabs be roughly adopted to the elastic theory ratios.

4. To limit the formation of cracks, the formulas given in the Netherlands Concrete Code - 1962, are prescribed. Here again, the yield stresses divided by the factor of safety are to be substituted for the allowable stresses.

2-A. C.U.R. (Committee of Concrete Research), "Experimental Investigations of the Plastic Behavior of Slabs", Rapport 26B, December 1963. (In Dutch with English Summary)

Results and Conclusions

Reports experimental results dealing with the yield moment in a section, the redistribution of bending moments, yield-line patterns appearing in various types of slabs and ultimate load of freely supported and restrained slabs. The membrane action is discussed, as well as the behavior of the slab under design load.

3-A. Enami, A., "The Investigation of Ultimate Load Analysis of Reinforced Concrete Prismatic Folded Plate Structures". World Conference of Shell Structures, San Francisco, 1962.

Results and Conclusions

The yield condition is based on Coulomb's internal friction theory (a modification of Mohr's theory) for concrete and the maximum stress theory for reinforcing bars in tension. The method is based on the lower bound theorem. A better statically admissible multiplier (load factor) is found by the equilibrium of applied loads and stresses at yield hinge lines.

4-A. Kemp, K. O., "A Lower Bound Solution to the Collapse of an Orthotropically Reinforced Slab on Simple Supports". Magazine of Concrete Research, July 1962.

Results and Conclusions

A lower bound solution to the collapse of a simple supported, rectangular slab, orthotropically reinforced and carrying a uniformly distributed load is developed. It is based on one originally proposed by Sawczuk, but the distribution of twisting moments has been modified to produce just positive yield moments at all points in the slab and negative yield moments at the corners.

The lower bound collapse loads calculated for a range of coefficients of orthotropy and ratio of width to length of slab, agree closely with the upper bound values derived from the yield-line theory. The yield lines, however, are quite different and are found to be an infinite set of curves in the lower bound solution.

Finally, the solution is used to determine the extent of negative reinforcement required in the corners of the slab and the loads transmitted to the supporting edges.

5-A. Sawczuk, A., "On Experimental Foundations of the Limit Analysis Theory of Reinforced Concrete Shells". Shell Research. Proceedings of the Symposium on Shell Research, Delft, 1961. Editors - Haas, A.M. and Bouma, A. L. North-Holland Publishing Company, Amsterdam.

Results and Conclusions

Analysis of collapse modes and types of failure of shells

may furnish a reasonable basis for a simplified method of the limit analysis of shells and yet within the fr. ework of the general theory of limit design. The general relations of the limit analysis theory are given for cylindrical vaults. A piecewise linear condition of failure for reinforced concrete shells is discussed. The possible discontinuities of the field quantities for cylindrical shell roofs are indicated. A simplified kinematic method is applied to obtain some qualitative data concerning the collapse load intensities for shells tested.

6-A. Hillerborg, A., "A Plastic Theory for the Design of Reinforced Concrete Slabs". Preliminary Publication International Association for Bridge and Structural Engineering, Sixth Congress, Stockholm, 1960.

Results and Conclusions

A load that is sufficiently great to cause failure of the plate through the formation of plastic hinges may be found by means of the yield line theory. This load is theoretically unsafe since other yield lines may form at a lower load.

The equilibrium theory states that if a distribution of moments can be found which satisfies the equilibrium equation and the boundary conditions for the plate under the action of a given load, and if these moments do not exceed the yield moments at any section of the plate, the plate is capable of carrying that load. The equilibrium theory gives a safe load. The exact load is somewhere between that calculated by the yield line theory and the equilibrium theory.

7-A. Hillerborg, A., "Theory of Equilibrium for Reinforced Concrete Slabs". Department of Scientific and Industrial Research, Building Research Station, Library Communication No. 1082, Great Britain.

Results and Conclusions

The equation of equilibrium of an element of slab with Timoshenko's notation is:

$$\frac{\partial^2 m_x}{\partial x^2} + \frac{\partial^2 m_y}{\partial y^2} - 2\frac{\partial^2 m_{xy}}{\partial x \partial y} = -q(x,y) \qquad (1)$$

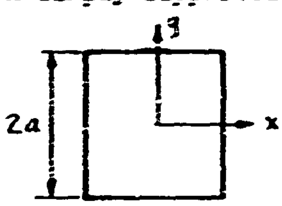
The three moments are independent of each other and therefore, two of these can be selected arbitrarily and the third moment solved by equation (1). The boundary conditions must also be satisfied.

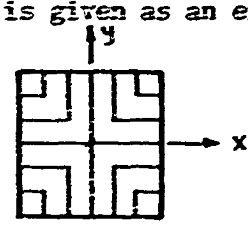
If, for a certain external load $q(x,y)$, a moment distribution can be found which satisfied eq. (1) and the edge conditions, and if the slab can take up these moments at each point, $q(x,y)$ is a lower limiting value for the bearing capacity of the slab.

The distribution of moments should not deviate too greatly from the probable distribution at the stage of failure.

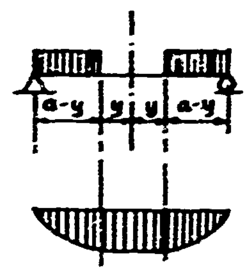
The most practical solution, the equilibrium method, is to divide the slab into strips in which the load in one strip is carried in one direction only.

A simply supported slab is given as an example.





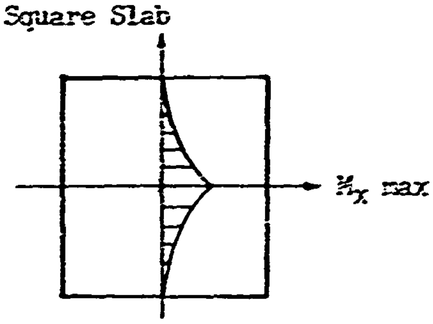
Strips



Simply Supported Square Slab

$$M_x, \max = \frac{q(a-y)^2}{2}$$

Loading and bending moments in a strip of slab at distance y from the x axis.



Distribution of maximum moments in strips parallel to the x axis.

$m_{xy} = 0$ in the first quadrant
$$\frac{\partial^2 m_x}{\partial x^2} = -q \quad \frac{\partial^2 m_y}{\partial y^2} = 0 \text{ for } x > y$$
$$\frac{\partial^2 m_x}{\partial x^2} = 0 \quad \frac{\partial^2 m_y}{\partial y^2} = -q \text{ for } x < y$$

The mean value of the maximum moments is $\frac{q\ a^2}{6}$

The strip method is also outlined for any slab supported along all edges and loaded by a distributed load and slabs with free edges and distributed load.

8-A. Gamble, W.L., Sozen, M.A., and Siess, C.P., "Measured and Theoretical Bending Moments in Reinforced Concrete Floor Slabs". University of Illinois, Civil Engineering Studies. Structural Research Series No. 246, June, 1962.

Object and Scope

The object was to develop a slab design procedure. Some tests of two-way slab structures reported in the literature were reviewed. Also studied were test results from the five tests of different 1/4-scale slab models tested at the University of Illinois.

Results and Conclusions

The yield line analysis was used to determine strength of slabs. Shear strength was calculated using Moe's formula, Elstner and Hognestad punching shear equations or equation from ACI Committee 326.

In nearly all of the tests reported in the literature, the failure loads are higher than the yield line analysis failure loads. Some reasons for this are:

1. Incorrect determination of steel strength.

2. Strain hardening of reinforcement.

3. Errors in establishing locations of yield line.

4. Arching of the load.

5. Effects of deformations of the structure.

In most of the five U of I slab tests, the flexural mode of failure was complicated by other distress, such as shear failure, beam-column distress, etc. However, it is indicated that if flexural failure of the structures had occurred, the yield-line failure load would be somewhat lower than the actual failure load.

For one flat plate which failed in shear, Moe's analysis

gave a load which was 9 percent less than the actual failure load.

9-A. Gamble, W.L., "An Experimental Investigation of the Strength and Behavior of a Prestressed Concrete Flat Plate". Commonwealth Scientific and Industrial Research Organization, Australia. Division of Building Research Report T8.0-9, 1964"

Object and Scope

Flat plate was nominally 3 inches thick and had three 9-ft. spans in one direction and two 12-ft. spans in the other. The lightweight concrete slab was axially prestressed in both directions, using 0.276 in. diameter high strength wire. The wires were straight and unbonded spaced at 4 inches in the direction of the 12-ft. spans and 6 inches in the direction of the 9-ft. spans. Also, a 2 x 2 ft. mat of welded wire fabric of 1/4-in. bars, spaced at 3 inches in each direction was placed over each column about 1 inch above the lower surface of the slab. Columns were 4 inches square.

Concrete strength ranged from 4020 to 4850 psi.

Loading was applied by air pressure within large plastic bags.

Deflection readings were made with surveyor's level sighting on steel rules.

Strains were measured with Huggenberger electrical resistance strain gages.

Load was applied in increments of 40 psf to failure load of 200 psf.

Results and Conclusions

Behavior was described as brittle since there was little deformation. The greatest deflection was in a 12-ft. span of 0.66 inch. Failure was by shear at four of the columns.

There was little cracking prior to failure, being mainly in the negative moment region over the columns.

The ultimate moments computed by yield-line analysis were:

12-ft. span	229 psf applied load
End 9-ft. span	290 psf applied load
Interior 9-ft. span	387 psf applied load

Dead load was 25 psf.

In calculating shear strength, the strength was assumed 30% greater due to benefits of prestressing.

Some results of shear comparisons were:

Moe's formula plus 30% minus 88% of failure load.

Elstner and Hognestad's formula (1956) plus 30% minus 80% of failure load.

10-A. Best, B.C. and Rowe, R.E., "Abnormal Loading on Composite Slab Bridges". Cement and Concrete Association, London. Research Report 7. October, 1959.

Object and Scope

Tests on three composite slab bridge models built to one-third scale are described. The bridges incorporated precast prestressed inverted T-beams placed side by side with cast-in-place concrete fill to form the slab. Quality of deck concrete was varied and mild-steel transverse reinforcement differed. The bridges were of 10-ft. span x 11 ft. -4 in. width and were 6 inches deep overall.

The first bridge had no transverse reinforcement. The second and third bridges had 1/4-in. diameter mild-steel reinforcement spaced at 8 inches over the central 4 feet and at 12 inches over the remaining span length. The third bridge had a lower strength concrete for the deck than the second bridge. The beams were post-tensioned with 0.2-in. diameter wires and grouted.

Ultimate loading was accomplished with four pairs of point loads centered within the span and width.

Results and Conclusions

The failure load was calculated by the yield-line analysis. For the first bridge, the theoretical value underestimated the actual value by less than 10 percent. The ratio of computed to actual ultimate loads for bridges 2 and 3 were 0.91 and 0.94 respectively.

11-A. Reynolds, G.C., "The Strength of Right Prestressed Concrete Slab Bridges with Edge Beams". Cement and Concrete Association, London. Technical Report TRA/237. December, 1956.

Object and Scope

Two similar prestressed concrete slab bridges were tested. The bridges were 4 ft. 7-1/2 in. wide with a 6-ft. 6-in. span and 2-1/2 inches thick. Edge beams in the direction of the span were 5-1/2 inches deep and about 3/4 inches wide. The slabs were post-tensioned and grouted in both directions.

The first b idge was simply supported at each end of the span but had all four corners held down. A concentrated load was applied at the center of the slab.

The second bridge was loaded at midspan but 9-3/4 inches off center and only the two corners farthest from the load were held down.

Results and Conclusions

Computation of the ultimate load by the yield-line theory overestimated the load for bridge 1 by 3 percent. For bridge 2, the measured load was 6 percent above the calculated load.

12-A. Reynolds, G.C., "The Strength of Prestressed Concrete Grillage Bridges". Cement and Concrete Association, Technical Report TRA/268, London, June 1957.

Object and Scope

"A method, based on plastic theory and using some simplifying assumptions, is described for the determination of the collapse load of grillage bridges. It is shown that it is possible to obtain an indication of the rotations which must occur at plastic hinges for full moment redistribution to take place. Nine small-scale bridges comprising a preliminary structure, six right bridges and two skew bridges have been tested to destruction. The results indicate that the proposed method of analysis slightly underestimates the ultimate strength of the bridges, and that for these structures it is not necessary to calculate the rotation at the plastic hinges."

In the theory it is assumed that in the main beams the bending moment and torque at failure are independent of each other and that in the transverse beams the torque is zero.

It was attempted, without success, to consider the open grillages as equivalent slabs so that a yield-line analysis could be performed.

13-A. Morice, P.B. and Reynolds, G.C., "The Strength of Simply-Supported Slab Bridges Subjected to Concentrated Loads". The Strength of Concrete Structures Symposium, London, May 1956.

Object and Scope

A number of tests are reported on both right and skew small-scale slabs of reinforced concrete. The slabs were normally reinforced, reinforced with wire fabric, or prestressed. They were simply supported and loaded by a concentrated load placed at or near the edge of the bridge.

The ultimate loads were calculated by the yield-line theory. The ratios of computed to measured load ranged from 0.92 to 1.31.

14-A. Christiansen, K.P., "The Effect of Membrane Stresses on the Ultimate Strength of the Interior Panel in a Reinforced Concrete Slab". The Structure Engineer, August 1963.

Object and Scope

In 1952, a series of load tests were carried out on a reinforced concrete building in Johannesburg. Two tests to destruction on interior slab panels showed the collapse load was more than twice that predicted by the yield-line method. The results were explained by an arching action due to the development of compressive membrane stresses in the concrete.

Results and Conclusions

Tests on four restrained and four unrestrained beams showed that an arching action does develop due to restraint.

Based on added strength above flexural capacity due to membrane action, the ultimate loads were again calculated for the building load tests. Considering membrane forces, the calculated loads were 94, 84 and 85 percent of the actual failing load.

15-A. Nielsen, M.P., "Exact Solutions in the Plastic Plate Theory". Bygningsstatiske Meddelelser, October 1963.

Object and Scope

This paper deals with the problem of finding exact solutions for the load-carrying capacity of plates following the Johansen yield condition. It is demonstrated that simple yield-line patterns with straight yield lines in many cases give the correct load-carrying capacity. Other solutions have been obtained transferring solutions from plastic plane strain theory (skip line theory). Some of the moment fields used may be of some practical interest, making the designer able to find at every point the necessary amount of reinforcement steel.

16-A. Ockleston, A.J., "Load Tests on a Three-Story Reinforced Concrete Building in Johannesburg". The Structural Engineer, pp. 304, London, October 1955.

Object and Scope

Describes load tests conducted on a reinforced concrete two-way slab building which was built in the 1940's. The building was to be razed so it was possible to carry loading to failure. Lateral loads were applied to small toilet and storage wings by jacking against another portion of the building. One-way and two-way slab panels were loaded with steel tie plates.

Results and Conclusions

Loading was not continued to complete collapse but the considered failure loads were in excess of yield-line theory predictions.

17-A. Ockleston, A.J., "Arching Action in Reinforced Concrete Slabs". The Structural Engineer, p. 197, June 1958.

Results and Conclusions

An earlier account of full-scale tests to destruction performed on a building scheduled for demolition reported that the load-carrying capacity of the reinforced concrete floor slabs was considerably greater than predictions based on plastic theories. Subsequent tests and further analysis produced a theory involving arching

action (compressive membrane stresses) of slabs spanning interior bays. This theory appeared to produce a satisfactory explanation for the excessive load capacity as measured by known plastic theories.

18-A. Ockleston, A.J., "Loading Tests on a Large Reinforced Concrete Slab Spanning in One Direction". The Concrete Association, Paper No. 5, April 1957.

19-A. Ockleston, A.J., "Loading Tests on Reinforced Concrete Slabs Spanning in Two Directions". Portland Cement Institute, Paper No. 6, Johannesburg, October 1958.

20-A. Donald Inspection Limited, "Report of Load Test of Typical Floor - Place Victoria for Place Victoria St. Jacques Co., Inc." Unpublished, 1962.

<u>Object and Scope</u>

Describes water load tests conducted on a waffle slab floor. The tests were made on a full-scale mock-up of a half-plan of the building. Two types of shear design were tested.

<u>Results and Conclusions</u>

The floor system failed in both instances in shear rather than flexure.

21-A. Guralnick, Sidney A. and LaFraugh, R.W., "Laboratory Study of a 45-foot Square Flat Plate Structure". Portland Cement Association, Bulletin D70, Proceedings Vol. 60, p. 1107, September 1963.

<u>Object and Scope</u>

Coordinated experimental and analytical studies of reinforced concrete floor systems were conducted at the University of Illinois and the Portland Cement Association laboratories for the purpose of providing a basis for more rational design methods than those now in use. Ultimately, it is expected that more economical floor systems will result from these improved design methods. The experimental program at the University of Illinois involved testing of one-quarter scale models of various floor systems.

To aid interpretation of the one-quarter scale model tests, a flat plate structure constructed at three-quarter scale and 45-ft. square was tested at the PCA laboratories. The distribution of moments in the slab found in the tests at service load is compared with values for slab moments obtained by current design methods. Also, the observed behavior at ultimate strength is compared with values for ultimate load predicted by application of the yield-line theory and of a shear strength theory.

22-A. Borges, J. Ferry, "Statistical Theories of Structural Similitude". Publication No. 164, Laboratoria Nacional De Engenharia Civil, Lisbon, 1961.

Object and Scope

Paper deals with statistical theories of structural similitude. Brittle rupture, ductite rupture and failure by deformation are treated.

Results and Conclusions

Increasing scale produces less statistical dispersion.

23-A. Lane, R.G.T. and Serafim, J. Laginha, "The Structural Design of Tang-e Soleyman Dam". Technical Paper No. 212, Laboratorio Nacional De Engenharia Civil, Lisbon, 1963.

Object and Scope

Structural design making use of models is described. Preliminary homogeneous plaster model studied first. Mercury used as loading mechanism. Heterogeneous plaster model of dam and foundation tested. Anticipated E values for concrete and rock were simulated. Model tested to failure with hydraulic rams. Plaster, plastic and microconcreted models subjected to vibration tests.

Results and Conclusions

Cost savings appreciable. Model testing best technique for designing arch dams of double curvature. Seismic study techniques still under development. Research in this field needed urgently.

24-A. Corley, W.G., Sozen, M.A. and Siess, C.P., "The Equivalent Frame Analysis for Reinforced Concrete Slabs". Civil Engineering Studies Structural Research Series No. 218, University of Illinois, Urbana, Illinois, June 1961.

25-A. Sozen, M.A., "Structural Damage Caused by the Skopje Earthquake of 1963". Civil Engineering Studies Structural Research Series No. 279, University of Illinois, Urbana, Illinois, January, 1964.

26-A. Jirsa, J.O., Sozen, M.A., and Siess, C.P., "The Effects of Pattern Loadings on Reinforced Concrete Floor Slabs". Civil Engineering Studies Structural Research Series No. 269, University of Illinois, Urbana, Illinois, July, 1963.

27-A. Woodring, R.E. and Siess, C.P., "An Analytical Study of Moments in Continuous Slabs Subjected to Concentrated Loads". Civil Engineering Studies Structural Research Series No. 264, University of Illinois, Urbana, Illinois, May, 1963.

28-A. Simmonds, S.H. and Siess, C.P., "Effects of Column Stiffness on the Moments in Two-Way Floor Slabs". Civil Engineering Studies Structural Research Series No. 253, University of Illinois, Urbana, Illinois, July, 1962.

29-A. Jirsa, J.A. Sozen, M.A. and Siess, C.P., "An Experimental Study of Flat Slab Floor Reinforced with Welded Wire Fabric". Civil Engineering Studies Structural Research Series No. 249, University of Illinois, Urbana, Illinois, June 1962.

30-A. Xanthakis, Manuel and Sozen, M.A., "An Experimental Study of Limit Design in Reinforced Concrete Flat Slabs". Civil Engineering Studies Structural Research Series No. 277, University of Illinois, Urbana, Illinois, December, 1963.

31-A. Vanderbilt, M.D., Sozen, M.A. and Siess, C.P., "An Experimental Study of Reinforced Concrete Two-Way Floor Slabs with Shallow Beams". Civil Engineering Studies Structural Research Series No. 228, University of Illinois, Urbana, Illinois, October 1961.

32-A. Gamble, W.L., Sozen, M.A. and Siess, C.P., "An Experimental Study of a Reinforced Concrete Two-Way Floor Slab". Civil Engineering Studies Structural Research Series No. 211, University of Illinois, Urbana, Illinois, June, 1961.

33-A. Ali, Iqbal and Kesler, Clyde E., "Rheology of Concrete -- A Review of Research". T. and A.M. Report No. 636, Department of Theoretical and Applied Mechanics, University of Illinois.

34-A. Atlas, Amos, Siess, C.P., Bianchini, A.R. and Kesler, Clyde E., "Behavior of Concrete Floor Slabs Reinforced with Welded Wire Fabric". T. and A.M. Report No. 260, Department of Theoretical and Applied Mechanics, University of Illinois.

35-A. Rocha, Manuel and Serafim, J. Laginha, "Rupture Studies on Arch Dams by Means of Models". Technical Paper No. 142, Laboratorio Nacional De Engenharia Civil, Lisbon, 1961.

36-A. Brotchie, John J., "General Elastic Analysis of Flat Slabs and Plates". ACI Journal, pp. 127-152, August, 1959.

37-A. Rosenthal, Israel, "Experimental Investigation of Flat Plate Floors". ACI Journal, pp. 153-166, August, 1959.

38-A. Rocha, M., Serafim, J.L. and Azevedo, M. Couz, "Special Problems of Concrete Dams Studied by Models". Technical Paper No. 155, Laboratorio Nacional De Engenharia Civil, Lisbon, 1963.

39-A. Hanson, N.W. and Carpenter, J.E., "Structural Model Testing - A Profile Platter". Development Department Bulletin D.68, Portland Cement Association Research and Development Laboratories, 1963.

40-A. Viner, W.C., Report of "Load Test of Typical Floor for Place Victoria". Donald Inspection, Limited, Montreal, October, 1962.

41-A. Elstner, Richard C. and Hognestad, Eivind, "Shearing Strength of Reinforced Concrete Slabs". Journal of the ACI, Vol. 53, pp. 29-58 July, 1956.

Object and Scope

Presented as a research report without practical design recommendations, this paper reports the methods and results of experimental work on the shearing strength of reinforced concrete slabs subjected to a centrally located, concentrated load. Tests of thirty-nine 6-ft. square slabs are reported. For 34 slabs, final failure was in shear by the column punching through the slab, in most cases after initial yielding of the tension reinforcement.

Major variables were: concrete strength, percentage of tension reinforcement, percentage of compression reinforcement, size of column, conditions of support and loading, distribution of tension reinforcement, and amount and position of shear reinforcement.

Results and Conclusions

The test findings show that the shearing strength of slabs is a function of concrete strength as well as several other variables. An ultimate strength theory was developed, by which the slab behavior under load may be explained and the measured ultimate loads may be predicted with satisfactory accuracy.

42-A. Abeles, P.W. and Lucas, J., "The Plastic Behavior of a Prestressed Concrete Shell With and Without Anti-twist Adjustments". Reprinted from "Non-Classical Shell Problems", Warsaw, September 2-5 1963. North Holland Publishing Company, Amsterdam - PWN - Polish Scientific Publishers, Warsaw, 1964.

43-A. Schlaich, Jong, "Dome Action in Continuous Reinforced Concrete Slabs". Stuggart Technical University Betong Und Stahlbetonbau, November - December 1964.

44-A. Janney, Jack R. and Wiss, John F., "Load-Deflection and Vibration Characteristics of a Multistory Precast Concrete Building". ACI Journal, Vol. 32, No. 10, 1961.

Object and Scope

The load-deflection and vibration characteristics of a structure are closely associated. The use of high strength concrete in prestressed building elements along with composite construction can produce building components which have resonant frequencies at various stages of loading to which the human body is sensitive. Vibrations with low amplitude may be sensed easily if the resonant frequency of the structure is relatively high. This paper describes full scale load and vibration tests conducted on a multistory precast building.

Results and Conclusions

The degree of composite action was determined and found to be much more complete than normally considered in design. The vibration characteristics were determined and are discussed.

45-A. Rostasy, Ferdinand S., "Connections in Precast Concrete Structures-Continuity in Double-T Floor Construction." Portland Cement Association Research and Development Laboratories Bulletin D55.

Object and Scope

This is a report of a laboratory investigation concerning continuity in precast-prestressed doublt-T floors. Continuity was established by placing intermediate grade deformed bars across the supports in a situ-cast topping, and by concreting the space between the adjacent ends of the double-tees to form transverse diaphragms.

The primary objective was to investigate the structural soundness of the continuity connection. Eight symmetrical double-T members 19 feet long were tested in negative bending. The prestress force and the amount of continuity steel were the main variables. In addition, four two-span continuous girders each having a total length of 64 feet, were tested to failure to investigate the applicability of limit design to this type of construction. The amount of design moment redistribution and prestress force were the principal variables in these tests.

Results and Conclusions

The tests demonstrated adequate strength and continuity behavior for the type of connection investigated. Use of limit design was found to lead to economies while still maintaining an adequate factor of safety.

46-A. Crawford, Robert E., "Limited Design of Reinforced Concrete Slabs". American Society of Civil Engineers, Journal of the Engineering Mechanics Division, Vol. 90, No. EMS, October 1964.

Object and Scope

Principles used in limit analysis of plates and slabs are introduced and examined briefly; application of these principles is illustrated by an example. The yield-line theory is considered in terms of limit analysis and is shown to give an upper bound on the collapse load of a reinforced concrete slab. Similarly, the equilibrium theory is reviewed and shown to give a lower bound on the collapse load of a slab. Results of the application of these theories to the limit design of complex slabs are outlined.

47-A. Gaston, J.R., Siess, C.P. and Newmark, N.M., "An Investigation of the Load-Deformation Characteristics of Reinforced Concrete Beams up to the Point of Failure". Civil Engineering Studies Structural Research Series No. 40, University of Illinois, Urbana, Illinois, December 1952.

48-A. Hognestad, E., Hanson, N.W. and McHenry, C., "Concrete Stress Distribution in Ultimate Strength Design". Portland Cement Association, Bulletins D6 and D6A (Discussion).

49-A. Lash, S.D., "Ultimate Strength and Cracking Resistance of Lightly Reinforced Beams". American Concrete Institute Journal (40-49), February 1953.

50-A. Lash, S.D., "Ultimate Strength of Reinforced Concrete Beams". American Concrete Institute Journal (29-46), February 1950.

51-A. Thoms, P.R., "Experimental Study of Beams in Elastic Foundations". American Society of Civil Engineers Proceedings, Vol. 86 (EM 3 No. 2505), 107-18, June 1960.

52-A. Benjamin, Jack R. and Williams, H.A., "Investigation of Shear Walls, Part 9. Continued experimental and Mathematical Studies of Reinforced Concrete Walled Vents under Static Loading". AFSWP 888, Stanford University.

53-A. Elstner, R.C. and Hognestad, E., "Laboratory Investigation of Rigid Frame Failure". American Concrete Institute Journal, pp. 637-668, January 1957.

54-A. Moody, Viest, Elstner and Hognestad, "Shear Strength of Reinforced Concrete Beams - Tests of Simple Beams". American Concrete Institute Journal, December 1954.

55-A. Moody, Viest, Elstner and Hognestad, "Shear Strength of Reinforced Concrete Beams - Tests of Restrained Beams with Web Reinforcement". American Concrete Institute Journal, February 1955.

56-A. Moody, Viest, Elstner and Hognestad, "Shear Strength of Reinforced Concrete Beams - Tests of Restrained Beams Without Web Reinforcement". American Concrete Institute Journal, 1955.

57-A. Elstner, R.C. and Hognestad, E., "Sustained Load Strength of Eccentrically Loaded Short Reinforced Concrete Columns". American Concrete Institute Journal, March, 1956.

58-A. Janney, J.R., Hognestad, E., and McHenry, D., "Ultimate Flexural Strength of Prestressed and Conventionally Reinforced Concrete Beams". American Concrete Institute Journal, Vol. 27, No. 6, February, 1956.

SELECTED BIBLIOGRAPHY

MODELS

LITERATURE REVIEW

UNPUBLISHED

1. Lecture Notes from MIT Special Summer Program, 1.52s, July, 1964.

PUBLISHED ARTICLES

2. Serafim, J. Laginha and Da Costa, J. Poole, "Materials and Methods for the Study in Model of the Stresses Due to the Weight in Dams". Publication No. 154, Laboratorio Nacional De Engenharia Civil, Lisbon, 1960.

    Object and Scope

    Determine stresses in a dam due to its weight including stresses at various stages of construction.

    Results and Conclusions

    Experimental methods for determining stresses are summarized; methods of inversion. immersion in heavy liquids, application of concentrated loads and stage construction are examined in more detail. Actual behavior as construction stages are completed can be determined experimentally.

3. Rocha, M. Da Silveira, A.F. and Azevedo, M.C. Cruz, "Note on Some Comparisons Between Experimental and Analytical Values of the Stresses and Displacements of Concrete Dams". Technical Paper No. 214, Laboratorio Nacional De Engenharia Civil, Lisbon, 1964.

    Object and Scope

    Comparisons made between analytical results and model results obtained for 5 arch dams and one buttress dam.

    Results and Conclusions

    Model analysis proved much more reliable than any of the analytical methods of predicting stresses. (Note: We do not know how the correct values were established.)

4. Borges, J. Ferry, Lima, C. Silva and Oliveria, E.R. Arantes E., "Matrix Analysis of Suspension Bridges". Technical Paper No. 213, Laboratorio Nacional De Engenharia Civil, Lisbon, 1964.

Object and Scope

Analytical definition of behavior of a suspension bridge cable is given; matrix analysis presented and computed results compared with values from model tests.

Results and Conclusions

Tests carried out on two 1/500 scale models in which towers and trusses were made of plastic and suspended cables made of steel wire. Computed influence lines and those from models investigation agreed very closely.

5. Serafim, J. Laginha and Azevedo, M. Cruz, "Methods in Use at the Lnec for the Stress Analysis in Models of Dams". Technical Paper No. 201, Laboratorio Nacional De Engenharia Civil, Lisbon, 1963.

Object and Scope

Methods and instruments in use at Lnec to determine stress fields in models of dams are described.

Results and Conclusions

Brittle lacquers are used to determine the directions of principal stresses; electrical resistance strain gages are used to obtain the magnitude of the principal stresses; and dial gages and twisted spring micrometers are used to measure deflections. Brands of instruments used are given, as well as techniques in their application.

6. Rocha, M., Serafim, J. Laginha, Cruz, A. Da and Cobeira, A., "Determination of Thermal Stresses in Arch Dams by Means of Models". Technical Paper No. 206, Laboratorio Nacional De Engenharia Civil, Lisbon, 1963.

Object and Scope

Study of thermal stresses in concrete dams.

Results and Conclusions

Mortar and plastic models of dams tested. Two techniques for producing temperature variations were oil baths on both sides of the dam with temperature controlled by coils in which hot or cold water was circulated and radiant heating with lamps and cooling with jets of air.

Temperature of the model was measured with thermocouples and strains were measured with electrical resistance strain gages.

The tests were conducted as computation methods and are not reliable for predicting thermal stresses.

7. I.S.M.E.S., 1951-1961, Instituto Sperimentale Modelli E Strutture. Brochure Instituto Sperimentale Modelli E Strutture, Bergamo, Italy, 1961.

Object and Scope

Describe facilities and model tests carried out at ISMES.

Results and Conclusions

Philosophy of models testing is presented as another means of extending engineering knowledge of structures.

The laboratory is designed to handle models testing by being equipped to make models, measure deflections and strains, calibrate and control gauging equipment, and assist on tests conducted outside the laboratory.

Thumbnail descriptions of techniques, materials and equipment used for testing models are presented, as well as brief descriptions of the various models tested during the first ten years of the laboratory's existence.

8. Rocha, M., Serafim, J.L. and Ferreira, M.J. Esteves, "The Determination of the Safety Factor of Arch Dams by Means of Models". Publication No. 163, Laboratorio Nacional De Engenharia Civil, Lisbon, 1961.

Object and Scope

Safety factor of dams determined by tests to failure of arch dam models.

Results and Conclusions

In tests it was determined that jacks applying a multiple number of paint loads are equivalent to loading by mercury. Factor of safety of the dams tested was above 10.

9. Swaminathan, K.V. and Prabhakara, M.K., "Vermiculite Mortar for the Preparation of Models for the Analysis of Concrete Structures" Indian Concrete Journal, 38 (1), pp. 6-10, January 1964.

Object and Scope

Investigate suitability of vermiculite-cement mortar for preparing models.

Results and Conclusions

A number of mortars were prepared, with and without an admixture of asbestos silicate of magnesia, with different aggregate-cement ratios and water-cement ratios. The stress-strain relationship is close in character to that of concrete, and the elastic characteristics can be varied within fairly wide limits. E ranged from 0.093×10^6 psi to 0.497×10^6 psi. f'_c ranged from 181 to 902 psi. Modulus of rupture ranged from 63 psi to 197 psi.

10. Sayskaya, T., "Waterproofing Electrical Resistance Strain Gages", ASTM Materials Research and Standards, Vol. 3, No. 4, April 1963.

Results and Conclusions

Gives formula for preparing rubber compound used to waterproof SR-4 gages on reinforcing steel.

11. Yates, J.G., Lucas, D.H. and Johnson, D.L., "Pulse-Excitation of Resistance Strain Gages for Dynamic Multi-Channel Observation". Proc. of Soc. for Experimental Stress Analysis - 11, No. 1, pp. 35-44, 1953.

Results and Conclusions

With pulses of a few microseconds through gages, it is possible to record several channels on a single cathode-ray tube.

12. McIntosh, J.D. and Erntnoy, H.C., "The Workability of Concrete Mixes with 3/8-in. Aggregates". Cement and Concrete Association Report 2.

Results and Conclusions

Experimental work to produce mix design tables for concrete mixes with 3/8-in. maximum size aggregate.

13. Davies, J.D., "Models for Structural Concrete". Civil Engineering and Public Works Review, April-May 1964.

Object and Scope

Present review of theory methods and materials for models of structure.

Results and Conclusions

I. Models to simulate behavior to collapse. Concrete can be simulated by "micro-concrete", pumice mixes and gypsum plaster. Can heat treat steel to obtain same stress-strain characteristics of prototype steel. If bond is a problem, as is likely with plaster, threaded rod may be used to assure that no slip occurs prior to failure.

II. Models to give data on elastic prototype structures. In elastic material Poisson's ratio may be a problem. Although elastic material may be a problem, it is better than using only analytic solutions, and the models can be used to estimate forces and moments rather than magnitudes and distributions of stresses and strains. Elastic materials usually used for indirect models tests.

14. "Automation for Model Test Readings", Anon. Civil Engineering and Public Works Review, May 1964.

Cement and Concrete Association Laboratory uses automatic strain logger purchased from Microcell Electronics, Ltd. Apparatus punches strain gage readout on tape for direct use on digital computer. Example - 250,000 strain readings obtained and processed within 15 weeks.

15. Fumagalli, E., "Suitable Materials for Static and Dynamic Tests on Model Concrete Dams". Cement and Concrete Association Library Translation No. 72, August 1958.

Object and Scope

Studies materials for models of dams and guide for the choice of materials to reproduce the characteristics of concrete.

Results and Conclusions

Homogeneous materials used consist of carrier material

with inactive fillers, such as rubber, ebonite, celluloid, etc.

Non-homogeneous materials used consist of a binder of cement, plaster or lime and inert granular fillers, such as pumice, sand, galena, rubber, cork, etc. For cement and pumice mortars Poisson's ratio 0.18 to 0.20, E can be adjusted between 2100 psi to 140,000 psi. Considered quite good to satisfy similarity conditions for behavior of concrete, both in tension, compression and E. But material has high shrinkage and requires "wet" type curing. Shrinkage reduced by removing pumice fines.

16. Russian Patent, "Mortar for Model Studies of Seismic Stability". Abstract in C.S. 59 (12) 1368a. December 9, 1963.

Results and Conclusions

The title material comprises a cement-sand mortar base. Rubber crumb and a gel-like aggregates are added to reduce modulus of elasticity.

17. Harris, H.G., Schwindt, P.C., Tahers, Imad, and Werner, S., "Techniques and Materials in the Modeling of Reinforced Concrete Structures under Dynamic Loads". MIT School of Engineering, Research Report R63-54, December 1963.

Results and Conclusions

The overall problem of reinforced micro-concrete models is examined from the point of view of clarifying the basis of model scale, aggregate gradation, size of test specimen and fabricating techniques. Discussion of problems and design of a dynamic loading machine to test small-scale models.

18. Moreno, A., "Lithargel, New Material for Elastic Tests on Reduced-Scale Models". Publication No. 121, Laboratorio Central de Ensayo de Materials de Construccion (Madrid), 1963.

19. Olivares, A.E., et al, "Load Distribution in Beam-Grille Bridge". Indian Concrete Journal, 37 (4), April 1963.

Results and Conclusions

Tests on 1:5 prestressed concrete scale model of bridge. Experimental results agree fairly with Gruyson-Massonet elastic analysis.

20. Rowe, R.E., "Experimental Methods in the Study of the Behavior of Shell Roofs". World Conference on Shell Structures. San Francisco, 1962.

Object and Scope

Review of some experimental research on the behavior of shell roofs.

Results and Conclusions

Plastics have been used and although there is concern of the difference of Poisson's ratio with that of concrete, very little conclusive evidence of the discrepancy caused by the difference in Poisson's ratio has so far been produced. There has been use of polyester resin with glass fibre cloth. This material has an E of 1.75×10^6 psi and Poisson's ratio of 0.15.

Gypsum plaster has been used for model making but requires that the plaster be not dried out or have additives to represent concrete's stress-strain relation. In connection with plaster it has been reported that threaded rod has been used to improve bond and more closely simulate behavior of the actual structure.

Microconcrete, sand-cement mortar, has been used extensively and reinforced and prestressed structures have been accurately represented.

Self-weight can be represented by many point loads or air on fluid pressure. When using the point-load technique, rubber rings have been inserted between the model and the loading frame in order that the same load is applied to each point.

Some of the techniques to measure deformation are photoelasticity, the Moire' method, electrical resistance strain gages, photostress and curvature measurements. Photostress developed by the Budd Company shows promise for use in the field as it is easy to apply the photoelastic material onto the structure.

21. Jones, L.L. and Base, G.D., "Test on a One-Twelfth Scale Model of the Dome Shell Roof for Smithfield Poultry Market". The Institution of Civil Engineer, Proceedings, January 1965.

Object and Scope

Test a reinforced mortar model of the elliptic paraboloid shell to determine magnitude of failure load and mode of failure.

Results and Conclusions

The shell thickness was 1/4 inch and required highest craftsmanship in fabrication. Reinforcement positioned by asbestos-cement spacers.

Uniform load over the whole shell or one-half of the shell was applied with pneumatic pressure of four rubberized fabric air bags. Because edge beams were prestressed, equivalent dead load was maintained at all times by continual operation of the compressor. Since leakage of air was small, excess pressure was bled off.

Strain was measured with electrical resistance strain gage rosettes and mechanical strain gages.

Data from the tests were processed on a digital computer and forwarded to the designer for use in the final design of the prototype. Failure was due to buckling.

Long-term effects were not investigated although the model was subjected to the equivalent dead load for four months.

22. Wright, P.J.F., "The Effect of the Method of Test on the Flexural Strength of Concrete". Magazine of Concrete Research, April 1953.

Object and Scope

An investigation was carried out to determine effect of size of specimen, method of loading and rate of load applications on the strength of concrete.

Results and Conclusions

Increasing the depth reduced the modulus of rupture approximately 30 percent.

The modulus of rupture under control loading was 20 to 25 percent higher than under third-point loading. But uniformity of results was better for third-point loading.

Increasing the load rate increased the modulus of rupture.

23. Little, W.A. and Hansen, R.J., "The Use of Models in Structural Design". Journal of the Boston Society of Civil Engineers, April 1963.

Object and Scope

Describe reasons for the basis of and techniques used in structural model analysis as currently practiced in the Laboratory for Structural Models at M.I.T.

Results and Conclusions

Buckingham's theorem is a means of determining the dimensionally dependent and independent quantities of a model. The analysis is the first step in setting up a model test.

For the behavior at failure loads under static load, concrete is used to represent itself in the model and the reinforcement is either steel or a phosphor bronze.

Fabrication of reinforced mortar models follows that of the prototype but at a miniaturized level.

Some of the suggested load systems are weights hung from discrete points, weight-lever systems, loading frames or air pressure. Strains are measured by either electrical or mechanical gages or photoelastic coatings. Deflections are measured by dial gages or photogrammetric techniques.

Interpretation of results must be preceded by an exhaustive study of possible sources of error, both random and systematic errors, as well as blunders. These sources of error should be discovered or eliminated during testing.

Examples of models tested are 1/80 scale models of shells made of a polyvinyl chloride material and 1/25 scale reinforced concrete dome structures.

It is mentioned that bond of the annealed small diameter wires is improved by rusting in a moist room.

24. Proceedings of a one-day meeting, "Model Testing". Cement and Concrete Association. Held in London, March 17, 1964.

Object and Scope

Present essential material for information of designers contemplating the use of models.

Results and Conclusions

Members of Cement and Concrete Association of staff presented lectures on various aspects of model testing.

Materials used should have same stress-strain relation as prototype, same Poisson's ratio, low modulus of elasticity and easy-to-form model.

Gypsum Plaster - can be tested 24-48 hours after casting. Within this period plaster has stress-strain relationship similar to that of concrete. However, the ratio of compressive to tensile strength is about 5 times that of concrete.

Cement Mortar - nearly identical to concrete and strength can be made in the range from 3,000 to 10,000 psi at 28 days.

Pumice Mortar - used at I.S.M.E.S. at Bergamo, Italy. If moisture is retained, modulus of elasticity is in the range of 2×10^3 to 1.4×10^5 psi. Poisson's ratio is 0.18 to 0.20. It has been shown that the material has the required similitude to concrete as regards both its stress-strain relationship and creep characteristics.

Measuring Techniques - Mechanical and electrical resistance strain gages, acoustic gages, photoelasticity, curvature gages and Moire' method.

Examples of correlation between behavior of models and prototypes showed excellent agreement. For Fleet bridge model and prototype, tests produced deflections and strains which were within 5 percent in both linear and non-linear ranges of behavior.

From models work on the Cumberland Basin Scheme, stress in concrete can be measured with a reliability of 7 psi. In the future it is expected to be able to automatically measure loads to about 1 percent, deflection to about 0.0001 inch, and rotation.

Model scale for the Manchester Skyway Bridge was chosen

because the least dimension would be 1/2 inch which is smallest practical size in view of placing reinforcement and casting.

For future studies of very complex structures, a number of highly instrumented, very small scale models could be made of an elastic material to aid in design and use only one larger mortar model without instrumentation to check ultimate load and mode of failure.

25. Biggs, J.M. and Hansen, R.J., "Model Techniques Used in Structural Engineering Research". RILEM Bulletin New Series No. 10, March 1961.

Results and Conclusions

Among the material presented are detailed diagrams of loading and deformation control and measuring systems.

26. Johnson, R.P., "Strength Tests on Scaled-down Concretes Suitable for Models, with a Note on Mix Design". Magazine of Concrete Research, March 1962.

Results and Conclusions

Mixes to 1/4, 1/8 and 1/16 scale of a mix with a 3/4 in. maximum aggregate were cast and tested. A 1-1/2 x 3 in. cylinder size gives satisfactory measurement of tensile and compressive strengths although results are more variable than those resulting from 6 x 12 in. cylinder tests. The smaller the scale of a mix, the lower the aggregate-cement ratio for a given water-cement ratio and workability.

The split cylinder tensile strength of 1/4 and 1/8 scale mixes is 9-14% of the compressive strength.

27. Neville, A.M., "Test on the Influence of the Properties of Cement on the Creep of Mortar." RILEM Bulletin No. 4, October 1959.

Results and Conclusions

It was found that creep is approximately proportional to the stress-strength ratio. Mortar specimens were subjected to constant compressive stress. Fifteen different portland cements were used.

28. Borges, J.F. and Lima, J.A.E., "Crack and Deformation Similitude in Reinforced Concrete". RILEM Bulletin No. 7, June 1960.

Results and Conclusions

It is shown that it is possible as a rule to reproduce conventional structures by making models to approximate scale with the same materials as the prototypes, i.e. concrete and steel. Phenomena of cracking deformation and rupture can be studied in the models obtained. This is confirmed by results presented which concerns bending tests of similar beams to different scales. Models were to 1/2.5 and 1/4 scales of full size beams.

29. Bignell, V.F., Smalley, V., and Roberts, N.P., "A New Photoelastic Material for Use in Problems Concerning Reinforced Concrete." Magazine of Concrete Research, November 1963.

Results and Conclusions

A modified epoxy resin reinforced with a magnesium alloy was used to fabricate rectangular beam specimens. Using photoelastic techniques, the stress pattern after cracking was observed.

30. Metz, G.A., "Verification of the Yield-Line Theory by Small-Scale Tests". Washington University Doctoral Dissertation, 1961.

Results and Conclusions

The yield-line theory is reviewed and some specific slabs are analyzed. The analysis is verified by testing small-scale models. Agreement between model test results and calculations was good.

31. "Beam Test Uses Compressed Air as Load", Anon. Engineering News Record, November 3, 1955.

Results and Conclusions

At the University of Alberta a uniform load was applied to a beam by using compressed air. Air pressure was along the bottom face of the beam acting upward.

32. Metz, G.A., "Flexural Failure Tests of Reinforced Concrete Slabs". Journal of the American Concrete Institute, January 1965.

Object and Scope

Sixteen small slabs were 7/8 inch thick and in plan ranged from 12 inches square to 24 x 36 inches. The slabs were reinforced mortar with the reinforcement consisting of two strands of soft black annealed wire twisted together so there were 2-1/2 full twists per inch. Twisting was done to provide adequate bond.

Results and Conclusions

The failure load was calculated using the yield-line theory. The measured failure load for all specimens was greater than the calculated failure load, ranging from 5 to 48 percent higher. Reasons for the higher measured strength were given as actual depth of reinforcement and concrete tension. Clear dimensions of the plan were used although the yield lines extend over the support and increased strength of the reinforcement beyond yield.

33. Nawy, E.G., "Flexural Cracking in Two-way Concrete Slabs Reinforced with High Strength Welded Wire Fabric". Journal of the American Concrete Institute, August 1964.

Object and Scope

Thirteen slabs 5'-8" square and 3.6 to 4.1 inches thick were tested. The concrete was reinforced with plain welded wire fabric. The wire size ranged from 0.192 to 0.33 inch in diameter. The steel was instrumented with SR-4 Type A-3 ERS gages. Concrete gages were SR-4 Type A-12 and AR-1 rosette gages.

Results and Conclusions

Crack width can be controlled by welded wire fabric. Orthogonal crack spacing in two-way slabs is a function of the orthogonal spacing of the welded reinforcement but only to an optimum limit of reinforcement spacing. This limit in the present investigation can be considered to range from 4 to 6 inches and active steel ratio of 0.75 to 1.2 percent.

34. Carlton, E.W. and Senne, J.H., "Instrumentation and Strain Measurement in Welded Wire Fabric Reinforced Concrete Slabs". Journal of the American Concrete Institute, pp. 141-152, October 1952.

Object and Scope

The 1/4-in. diameter wires were instrumented with A-7 SR-4 ERS gages. Gages were cemented to the wire using Duco cement and held in position by a small clamp, lined with sponge er, curved to the circumference of the wire.

The gages were waterproofed by layers of a synthetic rubber cement (3M Special Weather-strip Adhesive) and electricians rubber tape.

Load cell was made of 14-ST Aluminum 0.986 in. in diameter and 2.6 in. long. Two strain gages were mounted parallel and two at right angles to the axes of the cell. Cell was calibrated to 10,000 lbs. Sensitivity was 2 micro-inches for 12 lbs.

35. Self, M.W., "Ultimate Strength of Reinforced Concrete Flat Slabs". Journal of the Structural Division, Proceedings of ASCE, St. 4, August 1964.

Object and Scope

Three reinforced concrete slabs were tested. The models were 10 feet square, 2 inches thick, and supported by four interior columns producing an interior panel 5-1/2 square. Column supports were designed to rotate and translate freely, thus minimizing the influence of column restraint and membrane stresses. The steel reinforcement was 1/4-in. diameter cold rolled round rods. Uniform load was applied of an air pressure bag.

Results and Conclusions

The first slab failed in flexure at a load 9 percent above that computed by the yield-line theory. The failure of the second slab was described as combined shear and flexure. The ultimate load was 17 percent above the predicted yield-line strength for the second slab. The third slab failed in shear at a load almost identical to the predicted shear strength and only 4 percent above the computed flexural strength. Moe's equation for ultimate shear strength, as modified by Joint Committee 326, gave excellent predictions for the second and third slabs.

The strength above the predicted values for the first two slabs is attributed to strain hardening of the steel.

The computed flexural capacities were based on the assumption that the negative yield lines would occur in pairs and that they would be spaced at 0.444 times the diameter of the column capital. The actual yield patterns generally conformed to those assumed. If the negative yield lines had been assumed at or slightly beyond the face of the column capital, the predicted results would have been unconservative.

36. Park, R., "Tensile Membrane Behavior of Uniformly Loaded Rectangular Reinforced Concrete Slabs with Fully Restrained Edges". Magazine of Concrete Research, pp. 39-44, March 1964.

Object and Scope

Current test results, as well as another reported work, are presented on uniformly loaded rectangular slabs with edges restrained against translation. Slab sizes were 36 x 20.6 x 1.3 inches and 60 x 40 x 2 inches.

Results and Conclusions

A theory is developed in which the reinforcement acts as a plastic membrane. In the membrane action it is assumed the concrete is cracked throughout its depth and does not carry any load and the reinforcement has no strain hardening beyond yield.

In comparing the theoretical loads with the failure loads, the membrane theory gave conservative values; however, it is pointed out that the steel actually used had a tensile strength substantially above the yield stress.

37. Borges, J. Ferry, "Statistical Theories of Structural Similitude". Publication No. 164, Laboratoria Nacional De Engenharia Civil, Lisbon, 1961.

Object and Scope

Paper deals with statistical theories of structural similitude. Brittle rupture, ductile rupture, and failure by deformation are treated.

Results and Conclusions

Increasing scale produces less statistical dispersion.

38. Lane, R.G.T. and Serafim, J. Laginha, "The Structural Design of Tang-e Soleyman Dam". Technical Paper No. 212, Laboratorio Nacional De Engenharia Civil, Lisbon, 1963.

Object and Scope

Structural design making use of models is described. Preliminary homogeneous plaster model studied first. Mercury used as loading mechanism. Heterogeneous plaster model of dam and foundation tested. Anticipated E values for concrete and rock were simulated. Model tested to failure with hydraulic rams. Plaster, plastic and micro-concreted models subjected to vibration tests.

Results and Conclusions

Cost savings appreciable. Model testing best techniques for designing arch dams of double curvature. Seismic study techniques still under development. Research in this field needed urgently.

39. Rocha, Manuel and Serafim, J. Laginha, "Determination of Thermal Stresses in Arch Dams by Means of Models". Publication No. 133, Laboratorio Nacional De Engenharia Civil, Lisbon, 1960.

Object and Scope

Study of thermal stresses in dams.

Results and Conclusions

Mortar model of an arch dam subjected to temperature variations in order to determine strains induced by annual temperature variation. Model used since computation methods are not reliable for predicting thermal stresses.

40. Alami, Z.Y. and Ferguson, P.M., "Accuracy of Models Used in Research on Reinforced Concrete". American Concrete Institute Journal 60:1643-63, (Doctoral Dissertation, University of Texas), November 1963.

41. Schwaighofer, J., "Analysis of Structures by Aid of Models". Engineering Journal 46:22-6, Discussion 36, July 1963.

42. Ward, G.D. and Ely, J.F., "Apparatus for Maintaining Constant Loads on an Inelastically Failing Structure". Materials Research and Standards 3:730-3, September 1963.

43. Charlton, T.M., "Direct Method for Model Analysis of Structures". Civil Engineering Vol. 48, No. 559, pp. 51-53, London, January 1953.

44. Landeck, N.E., "Direct Method for Model Analysis". American Society of Civil Engineers Proceedings 82: (ST 1, No. 869): 1-15 January 1956. Discussion, J.F. Borges 82 (ST 4, No. 1024): 17-19 July 1956.

45. Brock, G., "Direct Models as an Aid to Reinforced Concrete Design". Engineering 187:468-70, April 1959.

46. Pletta, D.H. and Frederick, D., "Experimental Analysis". American Society of Civil Engineers Proceedings Vol. 79, Separate No. 224, July 1953.

47. Thurman, A.G. and Herman, G.J., "Model Studies of a Concrete Hyperbolic Paraboloid". American Society of Civil Engineers Proceedings 88: (ST 6, No. 3353) 161-81, December 1962.

48. "Models Make for Economy in Materials". Engineering 191:862-3, June 1961.

49. Roy, H.E.H. and Sozen, M.A., "Models to Simulate Response of Concrete to Multi-Axial Loading". Illinois University, Department of Civil Engineering, Structural Research Series No. 268, Urbana, Illinois, June 1963.

50. Beaujoint, N., "Similitude and Theory of Models". International

Union of Testing and Research Laboratories for Materials and Structures. RILEM Bulletin No. 7, pp. 14-39, June 1960.

51. Brown, E.H., "Size Effects in Models of Structures". Engineering 194:593-6, November 1962.

52. Durelli, A.J. and Daniel, I.M., "Structural Model Analysis by Means by Moire' Fringes". American Society of Civil Engineers Proceedings Vol. 86 (Journal Structures Division) No. ST 12, December 1960, Pt. 1, Paper 2693, pp. 93-102. Discussion 87 (ST 5, No. 2846) 95-8, June 1961. Reply 87 (ST 7, No. 2974); 281, October 1961.

53. Amaratunga, "Tests on Models of Concrete Structures". Engineering 194:62-3, July 1962.

54. Fumagelli, Doct. Ing. Emanuele, "Use of Models of Reinforced Concrete Structures". Reprint from Magazine of Concrete Research, July 1960. (Discussion, March 1961)

APPENDIX A

REPLIES TO QUESTIONS CONCERNING MICRO-CONCRETE MODELS

APPENDIX A

As part of our overall study, we formulated six questions and forwarded them to four of the outstanding model analysts in Europe. This communication was directed to the following men:

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Madrid, Spain

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Wexham Springs, Slough
Buckinghamsire, England

The questions and the corresponding answers are presented below:

1. What scale limitations do you feel should not be exceeded in studying reinforced concrete behavior by use of small-scale micro-concrete structural models?

<u>Rocha</u> As regards the scale of the models we consider that the limitations depend greatly of the type of phenomena to be studied. As for orders of magnitude, we can indicate 1/5 for studying cracking and 1/10 up to 1/20 for studying deformation and ultimate strength. Of course, very smaller scales of 1/100 up to 1/200 can be used for studying the elastic behavior in plastic models.

<u>Oberti</u> 1 : 1 to 1 : 15.

APPENDIX A

<u>Del Pozo</u> The scale of a micro-concrete structural model depends on a series of considerations. Among the most important ones, may we state the following:

(a) It is completely necessary to obtain in the model the same stress-strain rate that the one obtained in the real structure. Moreover, to study buckling, which has a paramount importance in some structures, as for instance shell structures, it is completely necessary to obtain in the materials, the same modulus of elasticity and of Poisson in the scale model that the ones occurring in the real structures.

(b) The minimum constructive thickness with micro-concrete in the scale model.

(c) Diameters and mechanical characteristics of commercial steel wires which represent in the scale-model the reinforcement of the real structure.

(d) The accuracy and sensibility of measuring instruments to be used on the scale model.

(e) There are some advantages in using models as small as possible inside of admissible limits of construction: tests result at a more economical price, it takes a smaller room, and tests are generally carried out in a shorter time.

In tests carried out, in the Laboratorio Central of Madrid, with shell structural models, the minimum thickness, which have been obtained, were between 12 and 4 mm. We think that 4 mm. should be the minimum value to be used and consequently, it must fix the scale limitation.

Our experience states that in that kind of tests in micro-concrete shell models, the scale used oscillate between 1 : 7 and 1 : 15, being the more used scale 1 : 10, due to size and easeness of construction.

For larger structures (beams, slabs, etc....), scales may be larger, and we have carried out tests up to 1 : 2.

<u>Rowe</u> The physical difficulties of manufacturing small-scale models in what we call micro-concrete are such that thicknesses of 0.2 in. represent the limit at the present time. Generally speaking, thin membranes, such as

are used for shell roofs, may be manufactured accurately and reliably in thicknesses of 1/4 in. even when the plan area covered by the membrane may be of the order of 18 ft. x 12 ft. In our more recent work we have found that when modelling a hyperbolic paraboloid cooling tower, that we think it possible to cast accurately and reliably a thickness of 0.2 in. and this represents the limit of our experience at the present time. I should perhaps add that for the thicknesses mentioned above there have always been included a minimum of two layers of reinforcement.

We find that the thicknesses of 0.25 to 0.2 can be controlled with an accuracy of probably ±15% in the worst cases and that this accuracy is comparable to that accepted in site casting.

Given the thicknesses mentioned above, the scale limitation can be defined and is, generally speaking, about 1/12 for shell roof models.

When simulating floor slabs where flexure is the predominant mode of behavior, 3/8 in. is probably a more practicable limit on the thickness.

2. What materials comprising the concrete mixture have produced the best results in your work?

<u>Rocha</u> In what concerns materials, we have been using micro-concretes in which the maximum size of the aggregate is reduced in proportion to the dimensions of the bars and the distance between reinforcements. Ordinary cement and sand are used and it is tried to choose a cement content and water-cement relation such as to have similar stress - deformation curves in the concretes of the prototype and of the model.

<u>Oberti</u> Pumice cement mortar.

<u>Del Pozo</u> Concerning micro-concrete which has been used with good results, it has been made with sand, cement and water, proportioned having into account the following:

(a) Sand has a grading of sizes to constitute a micro-concrete instead of a mortar with unique

aggregate size.

(b) The maximum size of the sand should not be greater than a quarter of the minimum thickness of the model.

(c) We have not employed maximum sizes of sand, smaller than 1mm.

(d) Ultimate strength in compression and in tension in micro-concrete, should be equal to the ones of the concrete which will be employed in the real structure. If the above condition is fulfilled, the modulus of elasticity and that of Poisson have similar values.

(e) The micro-concrete workability must be chosen in accordance with the construction method and with the characteristics of the reduced model, to obtain a good pouring of the same.

In this micro-concrete the steel reinforcements, reduced to scale, foreseen for the real work are to be fitted. Generally, the commercial wires of little diameter usually have an elastic limit and an ultimate load greater than those of the bars used as reinforcements for reinforced concrete. This, some times, makes necessary a heat treatment of the commercial wires to diminish the values of those characteristics and equal them to those of the reinforcements used in the real work.

Rowe

In manufacturing micro-concrete we use graded sand and ordinary Portland cement and the various fractions in the sand are sieved and re-combined to give the desired grading which is determined on the basis of the workability and strength required for the given mix. Generally speaking, we have not evolved a complete mix design procedure for micro-concrete, but we have established a number of standard mixes which, from experience, we have found to be useful in certain applications and to give certain strengths. Speaking generally, we have found that it is possible to use sand-cement ratios in the range from 2.5 to 4 with water-cement ratios in the range from 0.4 to 0.55; the sand sizes range from British Standard sieve size number 7 to 100 and quite frequently gap grading is

employed to improve the workability. For the range of mixes and water-cement ratios mentioned above, strengths of from 4,000 to 10,000 lb/in^2 can be obtained at 28 days; this strength is obtained from tests on a 4-in. cube.

It is difficult to make recommendations for a specific mix to be used for types of structures since this depends on many factors; however, the information which is given in the accompanying document would be of value in specifying mixes for similar projects.

3. What loading methods have you employed, and which do you prefer?

<u>Rocha</u> Loading methods depend on the type of problems to be tackled but systems based on weights and jacks are currently employed. Recently we have also carried out some dynamic tests, the seismic actions being represented by random vibrations.

<u>Oberti</u> Jacks.

<u>Del Pozo</u> Load application on the model must present a distribution similar to the actual one and at the same time it must allow for different and successive loading and unloading cycles in order to check readings in the dials.

The loading system must also permit the observation of the model faces to detect cracks development.

Most of the times it is more desirable to substitute on the model uniform loads for concentrated equivalent ones selecting the separation between loads so as to produce negligible local flexure (it is usually assumed to be about 7 to 10 cm.).

Loading system more frequently used have been the following:

(a) Using weights, previously checked and hanging from the model or acting on it through levers.

(b) Hydraulic jacks and the corresponding set of isostatic load distributors to ensure the load distribution in several equal loads.

(c) Small diameter cylindrical cans (9 cm. approx.) about 1.60 m. length acting as floats. The model is built over a 2m. deep water reservoir and the floating cans are hung from the model. With the reservoir completely full with water, the load on the model is null as the cylindrical cans float. Taking out water from the reservoir we increase uniformly and at the desired rate the load on the model. With this load system we may attain loads of 1.000 Kg/m^2, and filling with ballast the cans loads of about 1.700 Kg/m^2 can be reached.

(d) By means of pure latex rubber tubes previously checked, suspended from an end of the model and joined from the other end to a rigid frame, whose distance to the plan's support of the model may vary to one's wish acting on manual jacks, of those used for cars. The loads attained with this system have been approximately the same to those attained with method (c).

From the four systems we have just described, method (a) is employed when we have to load with small loads linear models such as beams or similar elements.

Method (b) has been employed in the test of plane models, such as: plates, one-way floor slabs, etc., which need loads relatively important. Jacks most commonly used are of 10, 25 and 50 tons maximum load.

Systems (c) and (d) are the most suitable when it is necessary to load surface models curbed or plane with uniformly distributed loads, such as shells. From these two methods, perhaps the latter would be preferable as it is more economical and not necessary to build the reservoir on which the scale model is to be placed.

<u>Rowe</u>

The loading methods which have been employed in model testing are: (i) multiple point loads with load applied by dead weight or jacks; (ii) the use of air bags.

As with the mixes, it is difficult to say that one method is to be preferred to the other; they both have their place depending upon the information to be obtained from the model test and the significance of

the method of loading on the stress distribution in critical regions. Air bag loading, or its equivalent water bag loading, while convenient in many respects suffers from the disadvantage that the specimen is obscured during testing and, therefore, important facets in the behavior may not be observed. However, when dealing with buckling phenomena, this type of loading is probably the only loading which can be used reliably.

The use of multiple point loads does entail elaborate setting-up procedures but once the techniques have been evolved for them they are probably the most widely applicable method for models of various types. We now use this method almost exclusively for all our model tests and we have found that by suitable positioning of the point loads, local concentrations of stress can be virtually eliminated.

4. Do you have any references to full-scale tests which have been conducted from which comparisons with model studies can be made?

<u>Rocha</u> It is difficult to present a duly selected list of references. Regarding the studies performed in Iberian Peninsula solely, we find that the works carried out by Eduardo Torroja about 20 years ago deserve to be mentioned, in particular those relative to "Fronton de Ricoletos".

<u>Oberti</u> Yes, from jobs carried out for a number of customers.

<u>Del Pozo</u> Most of the shells which have been tested in the Laboratory have been already carried out, although very few measurements have been done on the real structure.

In the case of plane superficial structures (plates, slabs, etc..) e.g. elements acting mainly supporting flexure stresses, we have more experience as we have tested 40 plates in scale model to compare with other tests carried out at full scale. One of the fundamental conclusions of these tests has been the important difference which exists in some cases between the behavior of the section on reinforced concrete under

flexure and its correspondent scale model. This is due to the hyper-resistance which some sections of reinforced concrete show. In that hyper-resistance have an important part the percentage of reinforcement, covering, etc.

In our case and with 1/2 scale model we have attained variations in relation with the test at full scale of about 20 and 25%. In my opinion when a test in scale model of this type of structures is to be carried out, the moments admitted by the sections at full scale and in the model should be experimentally determined by means of tests on beams.

Rowe — There are only a limited number of references pertaining to tests in which the opportunity has been available to test both model and prototype; some references on this aspect are given in references 10, 12 and 13 of lecture 1 in the attached booklet.

However, I think it is worth while pointing out that the validity of the use of models has been established by extensive work in other fields. I would draw your attention to the work of the Association in the problems associated with bridges for which elaborate theoretical analyses were evolved. Tests on hypothetical models were carried out for the bridges which showed the validity of the theoretical approach and then specific tests on actual bridge structures were carried out which also showed the validity of the analytical solution. One can gather from these, therefore, that the use of a model is a justifiable procedure for the design, in this case, of bridges, since both model and the prototype have the common ground of the theoretical analyses. This correlation is bound to obtain throughout the elastic range and I believe will also obtain right up to failure, as we have already shown in work on yield-line theory given in detail in Research Report No. 12 "The Ultimate Load of Simply Supported Skew Slab Bridges".

5. What type of instrumentation have you found most effective in evaluating the performance of mortar models through the entire range of loading to failure?

Rocha — The type of the instrumentation used is closely

connected with the dimensions of the model. In case of small-scale models, strain gauges are currently used; in case of larger models we employ mechanical strain meters with bases up to 40 cm. These movable instruments that employ balls connected to the model as references are especially useful due to their simplicity and reliability even in long-term tests.

Oberti — Two different kinds:

(a) for the elastic range the usual one, and

(b) beyond that range the recording type.

Del Pozo — The measurement instruments we have used in the test of micro-concrete scale models have been the following:

(a) Flexometers for the measurement of deflections. Sensibility 0,01 mm. and maximum range 1, 3 or 5 cm.

(b) Bubble clinometers for the measurement of rotations. Sensibility 0,0001 radians.

(c) Load cells of different capacities for the measurement of reactions and efforts on the model.

(d) Strain-gages for the measurement of strains. These, with the reservations due to the influence in the measurement of micro-cracking.

The above-mentioned instruments are easy to obtain in the commerce although the clinometers we have used have been constructed at the Laboratory starting from commercial bubble levels.

Another apparatus which is being proved is a "curvimeter" which permits to measure curvatures and consequently bending moments and has been designed using strain-gages incorporated to the apparatus.

We should not forget that the tests of reinforced or prestressed concrete structures in micro-concrete scale-models give accurate data on the behavior of the structure as far as cracking and failure is concerned, and it is rather usual to call them "failure models". Nevertheless, there can be also obtained data of the behavior of the structure during the elastic phase. That means that the measurement of deflections,

rotations, efforts and reactions are the fundamental ones and also the ones which are obtained with greater guarantee from the testing.

Rowe — The instrumentation used in any model test must always be related to the information that is required and, therefore, it is impossible to say which type of instrumentation is the most effective. Consideration must be given to the information required and appropriate instrumentation used to obtain that information.

We have used successfully electrical resistance strain gauges and mechanical strain gauges which are demountable or fixed to the specimen and both of these are perfectly acceptable for determining certain types of strain in specimens. Obviously the electrical resistance type is more suitable for purely elastic phenomena but it could nevertheless be used in regions of high strain where the micro-concrete itself is behaving in an inelastic manner.

Demountable mechanical gauges with small gauge lengths of 2 in. are particularly valuable in model testing and can be used reliably by skilled operators.

Probably the most important aspect of instrumentation is the necessity for methods of processing data from strain gauges of various types. This does imply that digital data logging techniques should be used wherever possible as a means of speeding-up the entire model test. This aspect is illustrated in lecture 2 of the enclosed booklet.

6. What thoughts do you have regarding the use of models to evaluate the behavior of reinforced concrete in those areas in which similitude principles are difficult to apply, such as creep, shrinkage and bond between reinforcing and concrete?

Rocha — The use of models in unexplored domains seems rather promising but it requires a basic research that was not yet carried out.

Soil and rock mechanics can be mentioned as instances of fields in which models are still of little use.

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Oberti — Each case requires a separate appropriate model.

Del Pozo — On this point I have no experience and I would prefer not to give an opinion.

Rowe — In our work so far we have found that as regards creep, micro-concrete can be regarded as having essentially the same creep characteristics as the normal concrete it simulates and, therefore, it is possible to simulate the effects of creep in a structure; this, of course, will only obtain assuming that the scale factor for stress for model and prototype is unity.

With regard to shrinkage, we have come to the conclusion that it is impossible to simulate this directly in the model; we therefore endeavor in all tests to eliminate so far as we can the effects of shrinkage on the model structure and to impose some external deformation specifically to obtain an assessment of the effects of shrinkage on the prototype.

The parameters governing shrinkage in actual structures are very difficult to define or specify and an assessment of shrinkage at best will only be very approximate and, therefore, I think the technique suggested above is the one which should be adopted in micro-concrete models.

With regard to bond I believe that it is possible to simulate bond conditions or, at least, to arrange that the cracking which is associated with bond can be simulated in small-scale members. We are at present working on this subject and those results which have been obtained previously by Ferguson and Ferry Borges have indicated that it is possible to do this within the required degree of accuracy. I am not saying that it is possible to completely reproduce the variability which is associated with cracking but that the general distribution of cracks in a member and, in particular, the width of cracks can be reproduced with a reasonable accuracy.

Security Classification

DOCUMENT CONTROL DATA - R&D

(Security classification of title, body of abstract and indexing annotation must be entered when the overall report is classified)

1 ORIGINATING ACTIVITY (Corporate author) Protective Structures Development Center Building 2590 Fort Belvoir, Virginia	2a REPORT SECURITY CLASSIFICATION Unclassified 2b GROUP

3 REPORT TITLE
A STUDY FOR THE DEVELOPMENT OF A STATIC LOAD TEST FACILITY TO BE ESTABLISHED AT THE PROTECTIVE STRUCTURES DEVELOPMENT CENTER, FORT BELVOIR, VIRGINIA

4 DESCRIPTIVE NOTES (Type of report and inclusive dates)
Final Report

5 AUTHOR(S) (Last name, first name, initial)
Wiss, Janney, Elstner and Associates
Des Plaines, Illinois

6. REPORT DATE	7a TOTAL NO. OF PAGES	7b. NO OF REFS
April 1966	126	112

8a. CONTRACT OR GRANT NO. DA-18-020-ENG-3566 b. PROJECT NO. OCD-OS-63-148 c. d.	9a ORIGINATOR'S REPORT NUMBER(S) PSDC-TR-18 9b OTHER REPORT NO(S) (Any other numbers that may be assigned this report)

10. AVAILABILITY/LIMITATION NOTICES
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11 SUPPLEMENTARY NOTES	12 SPONSORING MILITARY ACTIVITY Department of the Army Office of the Secretary of the Army Office of Civil Defense

13. ABSTRACT This report contains a review of the state of knowledge concerning inelastic behavior characteristics/ultimate load capacity of reinforced concrete floor/roof systems. Yield line theory normally predicts ultimate load less than tests show for systems that have perimeter restraint.

There have been very few tests of full-scale buildings to the collapse stage. Most of what ~~we~~ know today about behavior of two-way floor systems ~~has been~~ based on structural systems tested at less than full scale. The report concludes that less than full-scale testing is practicable. Models scaled down to 1/28 are feasible, best scale seems to be between 1/8 to 1/15. The report discusses materials, fabrication and loading techniques in setting up a model test program.

The report outlines a program in detail to test three-dimensional frameworks of conventional reinforced concrete floor and roof systems. The first step is to establish proper correlation between mortar models and full-scale prototypes on their principal behavior considerations: shear, flexural strength, bond and column action using mortar models of single elements. After correlation problems have been resolved, the program of testing three-dimensional frameworks can be carried out. Building frames are detailed for such typical floor systems as flat slab, flat plate, two-way slabs, pan joists systems, waffle slabs and precast prestressed concrete.

The report contains a bibliography of published articles dealing with ultimate strength of two-way and three-way systems and various theories that have been proposed. A review of papers dealing with structural model analysis is included.

DD FORM 1 JAN 64 1473

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14. KEY WORDS	LINK A		LINK B		LINK C	
	ROLE	WT	ROLE	WT	ROLE	WT
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Reinforced Concrete						
Structural Models						
Microconcrete						
Three-dimensional Models						
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